IRRIGATION PRACTICE
AND
ENGINEERING

VOLUME III

IRRIGATION STRUCTURES
AND
DISTRIBUTION SYSTEM

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PREFACE

Volumes II and III are essentially devoted to a presentation of the fundamental principles and problems of irrigation engineering. While the author has endeavored to meet specially the needs of teachers and students in technical schools, considerable descriptive information and cost data have been added for the purpose of making these volumes more valuable to the engineers engaged in the construction and operation of irrigation systems. For use as text-books in class-room work, some of the descriptive material and detailed information may be considered only briefly and more emphasis laid on the fundamental principles and on the problems of economic construction.

The preparation of these two volumes results in part from the development of a course in Irrigation engineering presented at the University of California. It is based on an acquaintance with a large number of irrigation systems, located in most of the States of the western part of the United States and in western Canada, obtained through many opportunities for examination of these projects and through connection with a number of them. The writer has not confined himself to his own experience and observations, but has discussed the principles of irrigation engineering presented in this work with a number of successful engineers, who have had much experience in the construction and operation of irrigation systems. He has also availed himself not only of contemporary literature pertaining to American Irrigation engineering, but has consulted a large collection of foreign publications, mostly from India, Egypt, Spain and France. While there is still considerable difference of opinion among engineers regarding some of the principles of design of irrigation works, it is believed that the opinions and principles presented are in accordance with correct theory and good practice as demonstrated by careful observation.

This treatise on irrigation engineering, as presented in Volumes II and III, is largely confined to canals and other works which pertain to the usual types of irrigation systems. No attempt has been made to discuss the subject of dams used for the development of storage, and of high masonry dams used for the

diversion of water. Excellent books on dams made it unnecessary and undesirable to include a brief presentation of this subject. On the other hand, much space has been devoted to a rather complete consideration of low dams used for diversion weirs.

The division of this work in two volumes has been made primarily to avoid an excessively bulky book in one volume. The division has had to be made more or less arbitrarily. Volume II, on The Conveyance of Water, begins with three chapters which pertain to irrigation engineering as a whole, and Volume III, on Irrigation Structures and Distribution System, contains chapters which are closely related to the conveyance of water. These two volumes are not entirely separate from Vol. I on Irrigation Practice, which has been presented as an introductory volume, and to which reference is made in Volumes II and III.

The author wishes to acknowledge his indebtedness to those who have aided him in the collection of the large amount of data and information used in the preparation of this work and to the large number of publications from which much valuable information has been obtained. Special acknowledgment is made to the engineers and managers of irrigation projects, who have so willingly made it possible for the writer to examine these projects under the most favorable conditions, and who have kindly furnished a large number of drawings and photographs, of which many have been selected for the illustrations of this work. The acquaintance with these engineers and managers has been a source of much satisfaction and encouragement, and the relations and interchange of opinions with them have resulted in a large measure in whatever may be the merits of this work. To the United States Reclamation Service thanks are especially given.

The tabulated references presented at the end of each chapter will serve in many cases as specific acknowledgment for the use of published articles.

The author will appreciate any suggestions for betterments and will be greatly obliged to any reader who will inform him of any errors which may have been overlooked.

B. A. Etcheverry.

Berkeley, California,
August, 1915.

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IRRIGATION STRUCTURES AND DISTRIBUTION SYSTEM

CHAPTER I

DIVERSION WORKS

GENERAL CONSIDERATIONS

Parts of Diversion Works.—The diversion works of a gravity irrigation system usually consist of a diversion weir across the river, canal headgates or regulator at the head of the canal, scouring sluices through the diversion weir, an overflow spillway or a wasteway in the canal just below the headgates and in some cases a fish ladder at one end of the weir and a logway through the weir.

The general layout of the different parts of diversion works is illustrated by the following examples:

Boise Project Diversion Works (Boise River, Idaho).—The general layout is shown in Plate I, Fig. A. The diversion weir proper, 216 feet in length, is of the Ogee type, built of concrete rubble with its crest 35 feet above the downstream floor. Its cross section is shown in Fig. 12C. The canal headgates are on the west bank of the river, and form eight gate openings, each 5 feet wide and 9 feet high. They are not at right angles to the axis of the diversion weir, as is usually considered desirable to keep a clear channel in front of the gates; but the relatively small amount of sediment carried by the water and the difficulties of construction presumably did not justify the greater cost of placing the headgates at right angles to the weir. At the west abutment of the dam is a logway 30 feet wide, and adjacent to it a fish ladder which is described further in detail. Next to the fishway toward the headgates are two scouring sluice tunnels, each 6 feet wide and 10 feet high, with their floor placed at about the same level as the downstream floor of the weir. Log booms, shown on the upstream side, direct logs toward the logway. The structure has since been modified by raising the crest of the weir with gates placed in a superstructure, and

by the installation of a power-house at the headgates to generate power for use in construction of a large storage dam, the Arrowrock dam, located farther upstream.

North Platte Project Diversion Works (North Platte River, Wyoming-Nebraska).—The general plan is shown in Fig. 1. The diversion weir is of the Ogee type (see Fig. 12B) and raises the river water level to divert it into two canals: the Interstate Canal, which has a capacity of 1,400 second-feet, and the Fort Laramie Canal, which remains for future construction. The canal headgates are at right angles to the weir and through the

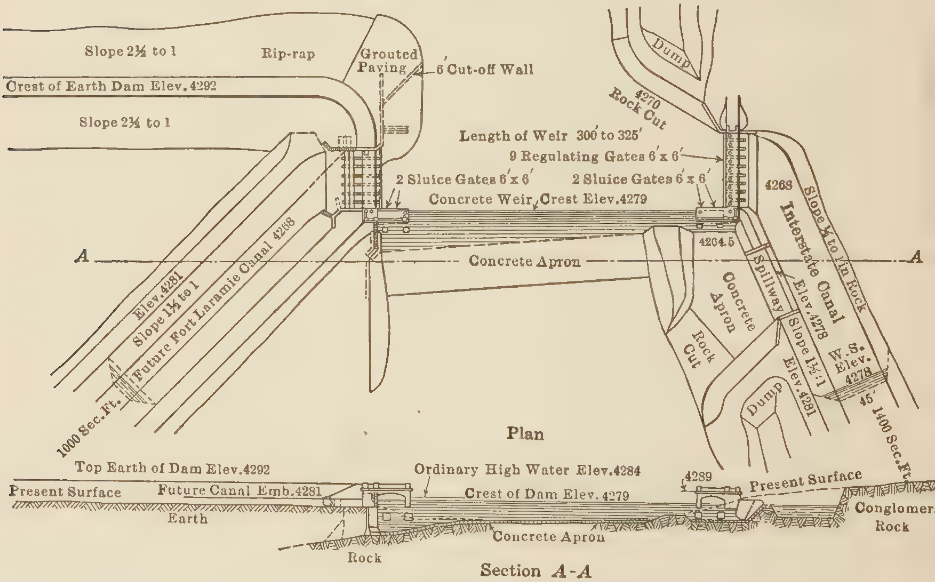

FIG. 1.—General plan of diversion dam on North Platte River and headworks for Interstate Canal. North Platte Project, Neb.-Wyo.

adjacent end of the weir, in front of each set of canal gates, two scouring sluices, 6 × 6 feet, are provided. Just below the headgates of the Interstate Canal an overpour spillway permits the overflow of excess water admitted into the canal. An earth embankment extends out from the Laramie end of the diversion weir to close the flood waterway of the river channel. The Interstate Canal headgates are shown and described in the discussion on headgates.

Salt River Project, Granite Reef Diversion Works (Arizona).—The general layout is shown in Fig. 2. The characteristic

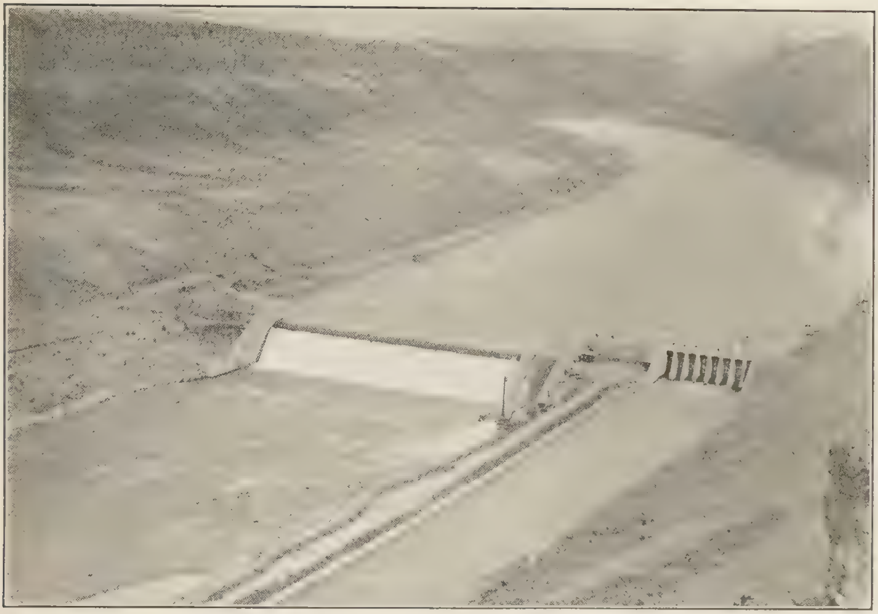

FIG. A.—Diversion works. Boise Project, Idaho.

FIG. B.—Diversion works. Sunnyside Project, Wash.

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PLATE I.

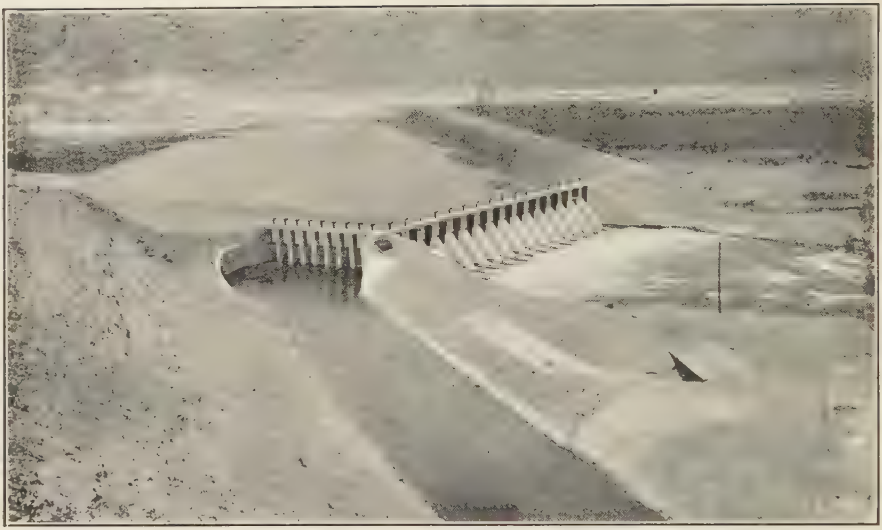


FIG. C.—Diversion works. Truckee Carson Project, Nev.

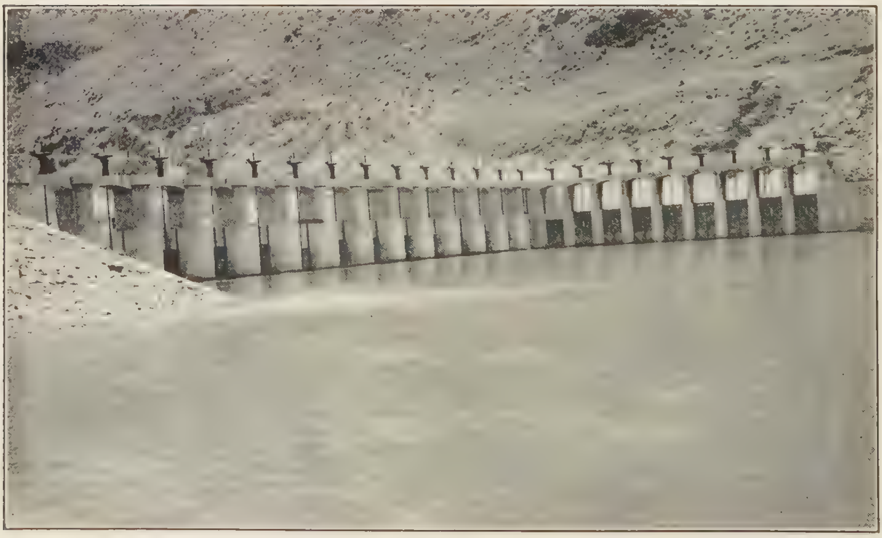


FIG. D.—Diversion works. Truckee Carson Project, Nev.

features are the provision of a sluiceway of large capacity at each end of the weir in front of the canal headgates. A brief description of the general layout is given in the discussion of the design and operation of the scouring sluices presented further.

Truckee-Carson Project Diversion Works on Truckee River (Nevada).—The diversion works consist of an open diversion weir with the canal headgates or regulator at right angles (Plate I, Figs. C and D). The diversion weir is 155 feet long and is divided by 15 piers, 5 feet thick, placed 10 feet on centers, into 16 gate openings (Fig. 3). The piers rest on a concrete foundation 8 feet thick and 30 feet wide, reinforced with 60-pound rails,

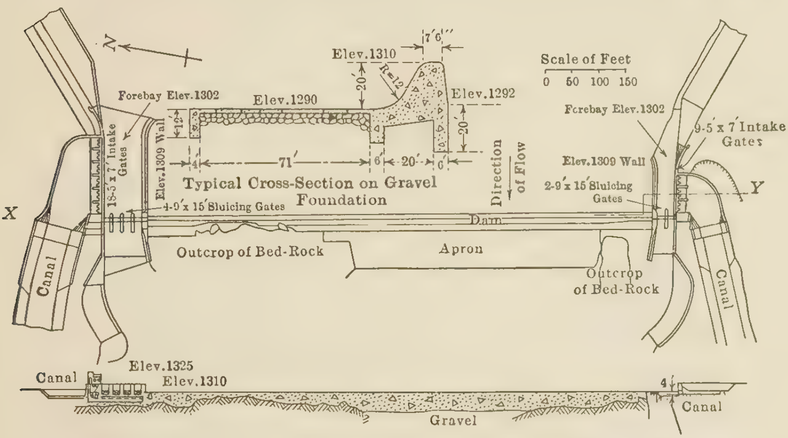


FIG. 2.—General layout of Granite Reef diversion works. (*Trans.* Am. Soc. C. E., Dec., 1914.)

spaced 5 feet on centers in both directions. Sheet steel piling 12 feet long was driven 2 feet from the upstream face of the foundation floor for a distance of 185 feet. These piles extend well in the bed of the stream, which is composed of gravel boulders and sand. Beyond the concrete foundation a paving of large stones extends 30 feet downstream; the thickness of the paving is 5 feet at the upper end and 2 feet at the lower end. The piers are walls 15 feet high, 6 feet 3 inches wide at the top 18 feet wide at the bottom, and 5 feet thick. These walls are not reinforced, and are designed as gravity walls. On top of these piers are smaller piers on which rest the operating platform. The operating platform is arched and reinforced with I-beams well anchored to the pier to resist the upward pressure when forcing the gates closed.

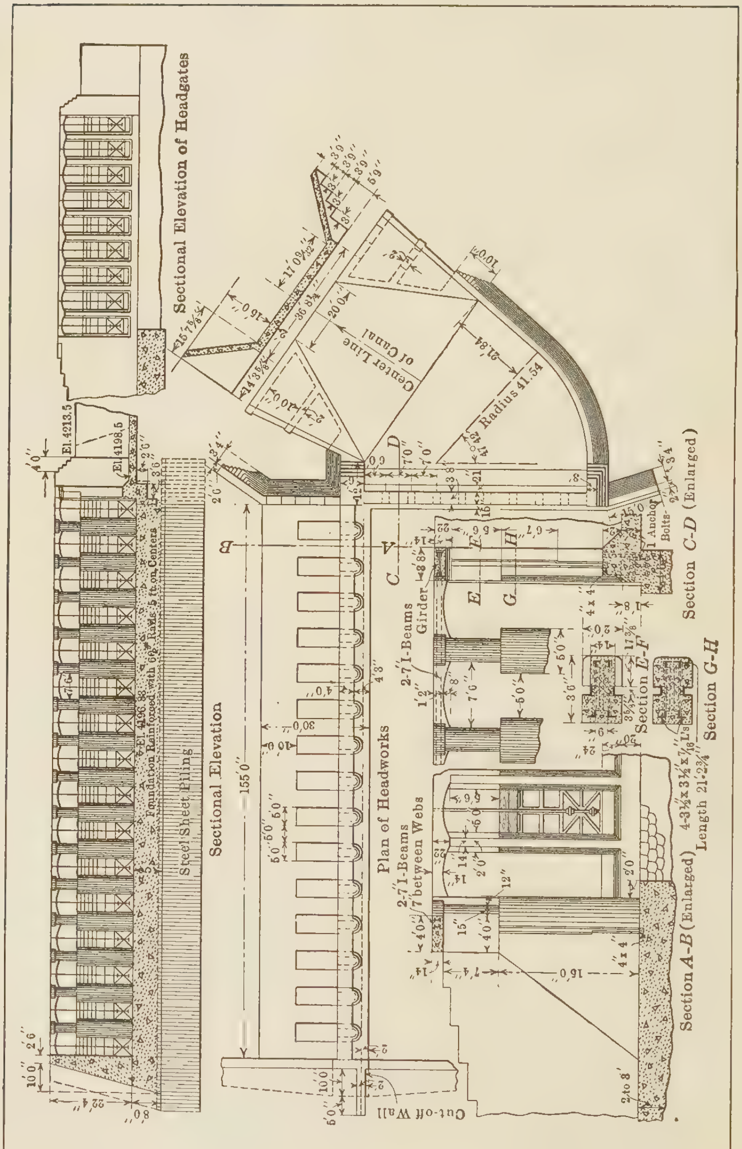


FIG. 3.—Diversion weir and headgates. Truckee Carson Project, Nev.

The gates for each opening consist of a set of double sectional cast-iron gates, each 5 feet high, and a set of flashboards for the upper 5 feet. When the lower gate is raised 4¾ feet it catches the second cast-iron gate and both are raised together. The total height of opening with gates fully raised is about 15 feet. The advantage of having the gate in sections is that it does not require as large a lifting force to raise them as if they were in one piece. To reduce friction all bearing surfaces are machine faced. The lifting stand is of cast-iron with bevel-gearing operated by a hand screw. The headgates consist of nine gate openings 5 feet wide. The posts between are 7 feet center to center and 2 feet thick. For sluicing the silt and sand past the headgates, the sill of the headgates is made 3⅔ feet above the floor of the weir. The concrete floor has an average thickness of 2 feet.

The headgates differ from the weir in that instead of piers of plain concrete the posts are made of concrete reinforced with built-up steel girders which allow of much lighter construction. The posts are designed as beams, such that about two-thirds of the water pressure is transmitted to the floor and about one-third to the operating platform, which is reinforced with a built-up steel girder to resist this pressure. The gates are similar to those of the weir, except that there are only two flashboards instead of five, and the total height of the gates is 11 feet 4 inches.

Beyond the headgate is an overflow spillway, with its crest 13 feet above the bottom of the canal, intended to dispose of excess water admitted in the canal.

The total cost of these headworks was $85,390 distributed about as follows:

Cement, 3,971 barrels at $2.95	$10,126
Excavation	13,397
Concrete, 3,322 cubic yards	19,932
Sheet piling	5,265
Gates, guides, stands, etc.	12,129
Riprap	13,548
Temporary flume	2,373
Steel girders, lumber, puddling	8,520
	$85,390

On the opposite side of the river from the canal headgates the river bank is low and the land surface is below the flood plane.

This part of the flood waterway is closed with an earth embankment which connects with the weir abutment.

This type of open diversion weir, with permanent piers forming narrow openings, is not well adapted to streams which during flood flow carry large floating material, and on this project an unusual flood, which carried down with it trees and large floating material, resulted in the obstruction of the openings and produced considerable damage. Since then the diversion weir has been modified by removing some of the intermediate piers in order to form wider openings to be better prepared for such floods.

Headworks of Yakima-Sunnyside Project, Washington—(Plate I, Fig. B).—The diversion weir is a closed weir of the Ogee type. The headgates are separated from the weir by the gate tender's house. About 25 feet away from the gate tender's house is a scouring sluice 6 feet wide, which is too small and not placed near enough to the headgates to be effective. The stream, however, carries little silt, so that the opening may have been provided to facilitate construction or for the passage of logs (Fig. 4).

The weir is 500 feet long and rests on rock (Fig. 12A). The width of the weir from the upstream face to the end of apron is 20 feet. The height of the crest above the apron is 7½ feet. In the deeper portions of the stream where the crest is more than 7½ feet above the bed of the river, the apron is continued with a series of steps 1 foot 6 inches wide, and 1 foot high. Cut-off walls 2 feet wide extend 1 foot into solid rock at the upstream toe of the weir and at the lower end of the apron. The crest of the weir is 6½ feet above the sill of the headgates, which assures that depth of water at the gates when the water just overflows over the weir.

The headgate structure is 43 feet 6 inches wide and 19 feet 6 inches high. The top of the operating platform is at the same height as the top of the earth embankment beyond the end of the diversion weir. The headgate consists of six gate openings 6 feet high and 6 feet wide, separated by buttress walls 18 inches thick. Between the buttress walls above the gate opening are panel walls 8¼ inches thick reinforced with ⅜ inch square bars.

The operating platform is 8 inches thick and is reinforced with ⅜-inch steel square bars, placed 6 inches on center 1½ inches from the bottom of slab. The gates are vertical cast-iron gates

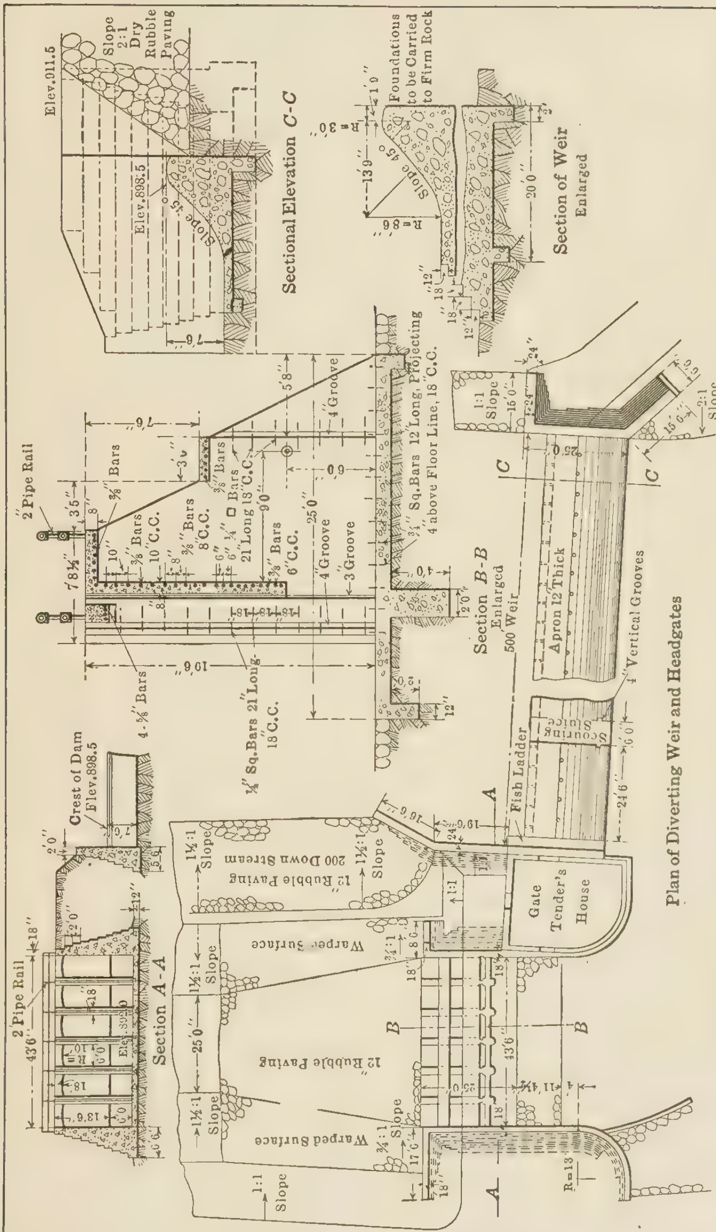


FIG. 4.—Diversion Works. Yakima-Sunnyside Project, Washington.

with $3\frac{1}{2}$-inch gate stems. They slide in metal grooves placed in the buttress walls and are lifted by means of cast-iron lifting stands with beveled gear operated by hand.

Below the cast-iron gates is another set of wooden gates of the Taintor type, which are radial gates hinged 6 feet above the floor and 9 feet from the outside face of the panel wall. These gates are emergency gates to be used when necessary to close the gates rapidly, but would very seldom be used.

The dam was built of rubble concrete composed of large stones, which could be handled by one man, incorporated in the concrete. The proportion used was 43 per cent. rubble stone and 57 per cent. concrete. Eighteen hundred and nine cubic yards of rubble concrete were put in the dam at an average cost of $6.40 per cubic yard, excluding engineering and administration. For the headgates, 493 yards of concrete were put in for the foundations and wings at an average cost of $13.95 and 38 yards for the buttress walls, panel walls and platform at an average cost of $28.70, including reinforcement. The cast-iron gates cost $2,529 in place. The concrete work for the headworks cost nearly $20,000. Excavation for abutments and dam, building of dyke, widening of river channel, excavating main canal, etc., brought up the total cost of the headworks to $48,531, including administration.

DIVERSION WEIRS

Position and Height of Diversion Weir.—The weir is generally placed at right angles to the direction of flow. A skew weir has a tendency to cause currents parallel to the weir axis, which on soft foundation may result disastrously. A skew weir should only be used on firm foundation.

The main object of a diversion weir is to raise the water level in the river to divert the desired flow in the canal through the headgates. The required height of weir to produce this must be determined from a consideration of the stream flow during the period of low flow. On many irrigation systems during part of the irrigation season at least the stream flow may be only about sufficient or less than the desired flow in the canal, so that there will be little or no surplus water pouring over the crest of the dam. It is necessary that the crest of the weir or the minimum low water level over the weir be built or raised to a minimum eleva-

tion above the full supply water level in the canal by a difference equal to the head necessary to give the desired velocity through the headgate openings. This head will usually be small, for it is generally more economical to use a small velocity and large gate openings than it would be to increase the height of the diversion dam in order to obtain a higher velocity and correspondingly smaller gate openings. When the entire stream flow has to be diverted through the headgates, a crest elevation 6 inches above full supply water level in the canal will usually be sufficient. Considering the flow through the gate openings as submerged and with a coefficient of discharge of 0.8, the discharge per square foot of gate opening for a 6-inch head is $Q = CA\sqrt{2gh} = 0.8 \times 1 \times \sqrt{64.4 \times \frac{1}{2}} = 4.55$ cubic feet per second.

The elevation of the crest of the diversion dam is determined from the elevation of the full supply water level in the canal as stated above, and the height of the weir at any point is determined from the height of the water surface in the canal above the stream bed. When the canal is located along the side hill or bank of the river, the grade of the canal being flatter than that of the river, it is possible to continue the canal upstream to a point where the water level in the canal would be lower than the elevation of the minimum low water level in the stream, in which case it may not be necessary to construct a diversion weir. This, however, is only feasible when the minimum stream flow is considerably larger than the desired flow in the canal, and is not the usual case. Below this point of intersection of the canal line with the stream there may be several points in the river channel which offer favorable conditions for the construction of a diversion weir; the choice will then be between a comparatively high weir and shorter diversion line with a comparatively low weir and longer diversion line. The choice should be based on a consideration of total cost, of operation and maintenance cost, of extra seepage losses in the longer line, etc.

On some projects an economic problem of a little different character occurs, where the stream flow is to be diverted from the river after it emerges from the hills into the valley land or through benches. The river may have cut a moderately deep channel so that its bed at the selected point of diversion is considerably below the surface of the bench or valley land. To

divert the water the choice may then be between a low diversion dam with a canal in deep cut and a higher diversion dam with a canal whose bed will be nearer the ground surface. For the deeper canal the available fall on the canal line, through the bench or valley, must give sufficient excess grade to be able to start the canal in deep cut at the headgates and gradually bring it out to an ordinary cut and fill section. The most economic solution requires a cost comparison by balancing the increased cost of canal excavation obtained with the deeper canal against the increased cost of a higher diversion weir.

An additional consideration, of great importance in determining the height and character of the diversion weir when it is to be constructed on the valley portion of the stream, is its effects on the lands above it. The weir, especially during flood flow, may produce a rise in the normal water level, which will make the river overflow its banks, in some cases flooding considerable valuable lands on the upstream side or causing a tendency for the river to acquire a new channel through some depression or into an old channel, or around one end of the diversion weir. This is prevented by training or protection works such as levees to confine the river in its channel, or by continuing the diversion dam proper at one or each end with an earth embankment on the axis of dam of sufficient height to be safe against overtopping and extending out until it tapers into the land surface sloping toward the stream channel. It may also require an open weir with collapsible crest to allow the flood flow to pass on with the least obstruction.

Length of Diversion Weir.—The length of the weir will also affect the height to which the water will be backed up on the upstream side. For some of the conditions indicated above the weir site will often permit a selection of the weir length within a certain range. It may be desirable to use a long weir and decrease the length of the embankment which extends into the side hill, or it may be preferable to confine the weir to a shorter length, increasing the length of the embankment. Where several weir sites are available a larger selection in weir length may be possible. A short weir will have to be designed stronger on account of the greater depth of water and the greater strength required to resist greater dynamic forces. On the other hand, where a stream has a slow velocity and carries considerable sand or silt, it is preferable to restrict the weir length to as short a length as feasible in order

to obtain a higher velocity and prevent the deposition of silt which in some cases may form islands above the weir, causing considerable trouble by diverting the channel of the river away from headgates. As a rule it is preferable to use the weir site giving the shortest weir. Where the overflow of valuable land may occur, the extent of flooded lands should be carefully considered; this may require a study of the backwater curve, according to the principles and computations given in the standard books on Hydraulics.

Location of Diversion Weir.—These considerations show that the location of the weir will be determined from the position of the canal and from a comparison of the conditions at certain points on the river which are favorable to the construction of the headworks and upper part of the diversion canal.

The location may be made by three methods:

First.—By maps obtained from topographic surveys of the entire country, using when available the topographic maps of the United States Geological Survey. This will usually permit only an approximate location, but may give the basis for the location by the other two methods.

Second.—By leveling from the highest point of the land to be irrigated, allowing the proper canal grade and continuing the level line until it intersects the stream, or up to a selected point of diversion. To avoid obstacles and permit more favorable canal construction, the canal line may be moved up on higher benches or extra grade introduced which will be taken up by the use of falls or chutes.

Third.—By selecting suitable diversion points on the river, which are known to be at an elevation sufficiently high to command all the land to be irrigated, and running from this point the level lines and location lines down to the highest point of the area to be irrigated, allowing the proper grade and introducing falls or chutes to use the excessive grades at points which will be favorable to economic canal location.

General Character and Types of Weir Locations.—The location of a weir site will be at one of the following points on the stream: (1) Junction of plain and foothills. (2) In plain or valley. (3) In the hills.

1. When the weir is located on the stream where it emerges from the hills, at the junction of the valley land with the foothills, the following conditions are generally encountered: The

subsurface of the stream channel will usually be satisfactory for good foundations as it generally consists of coarse sands, gravel and boulders, which in some cases overlay bedrock at a moderate depth. The surface topography along the line of the diversion canal will be fairly smooth, the soil firm and few if any deep drainage channels will have to be crossed. These conditions are favorable to an inexpensive diversion canal.

2. When the weir site is on the valley portion of the stream, the subsurface of the stream channel is usually gravel, sand or clay and unless a clay stratum or impervious bed can be reached by sheet piling or a cut-off wall, there will be more or less underflow under the dam. The surface topography is usually smooth and fairly level and the stream has a flat grade. These conditions are favorable to a short and inexpensive line and to a low but long diversion weir. The disadvantages are: (*a*) that the stream may not have sufficient fall to permit an easy diversion; (*b*) that there is danger of submerging land above the weir and of the stream cutting a new channel; (*c*) on streams with wide sandy beds, islands tend to form above the weir and may cause erosive currents parallel to the weir axis. These effects are largely prevented by using either an open weir or a closed weir with removable crest.

3. When the weir site is in the hills the subsurface of the stream channel is solid rock or boulders. The surface topography is rough, the slopes of the side hills are steep, and the grade of the river large. These conditions frequently require a strong weir in a narrow gorge and difficult and expensive construction of headworks and diversion line, with frequent cross drainage works, flumes, siphons, culverts, etc.

Conditions Favorable to a Good Site for Headworks.—When several sites are available the best one is that which satisfies the most of the following requirements:

1. Where there is room for the construction of canal headgates and diversion line without the necessity of expensive construction such as tunnel work, retaining wall sections, deep cut, rock excavation, etc.

2. Where the canal headgates can be placed at right angles to the weir so as to maintain a clear channel in front of the gates.

3. Near suitable building material.

4. Where good foundation and permanent banks can be obtained.

5. Where an impervious stratum is at or near the surface of stream bed.

6. Where the grade of the stream is steep enough so that with a low weir the canal and stream shall be near the same level only for a short distance.

7. Where an expensive high dam will not be needed to divert the water. It may, however, be cheaper to use a high dam than to use a longer diversion line.

8. Where the construction of the weir will not cause flooding of valuable lands above or tend to change the stream channel.

9. Where the velocity in the stream shall be preferably less than the velocity in the canal, to prevent silt deposits in the canals.

10. Where the stream channel is straight with uniform velocity and regular cross section. If on curved channel, when the velocity of stream is small, the headgate should be placed on the outside or concave side of the channel; when the velocity is great, and especially with soft banks, the headgate should be placed on the inside or convex side of the curve, where it will be safe from erosion. This requirement is more important for sites where no diversion weir is necessary.

CLASSES OF DIVERSION WEIRS

Diversion weirs belong to two general classes: closed weirs and open weirs:

Closed weirs are usually comparatively low dams built as complete obstructions across the river so that the entire stream flow passes over them. To this class belong: Brush and cobble weirs, log weirs, pile weirs, crib weirs: either continuous or built up of separate cribs, solid concrete or masonry gravity type weirs, loose rock weirs of the type used in India, and framed weirs of wood, steel or reinforced concrete.

Open weirs are built across the river to produce the least obstruction to the flow. The stream channel is usually divided into a number of openings or bays, separated by piers, columns, or framed buttresses of wood, steel, or concrete, supporting at the top the operating platform. The openings are closed and regulated by horizontal flashboards for the simplest type, or by lift gates of wood or steel, and in some cases collapsible gates. The diversion weir may have to be designed to give an entirely

unobstructed channel, such as when considerable large floating material is carried during flood time, in which case collapsible or removable gates are used and the piers and platform may be entirely omitted or may be made in sections either loose or hinged, so that they can be removed or collapsed with the gates during flood flow.

The selection of the type of weir will depend on the character of stream flow, the effect of the weir on the flooding of lands above, the character of the foundation, the permanency desired and amount of money available, and the availability of the material of construction. An open weir is to be used in preference to a closed one where the obstruction formed by a closed weir would raise the flood height of the water level to such a height as to cause the flooding of valuable agricultural land; or would result in the formation of sand or gravel islands on the upstream side of the weir. Closed weirs are self-acting, do not require the operation of gates, and will permit the passage of ice, trees and other floating material.

DESIGN OF DIVERSION WEIRS

The design of diversion weirs will depend on the type of weir and the local conditions, but there are static and dynamic forces common to all types of weirs, and general principles of design may be stated which are applicable to any type of weir. These principles will be derived from a consideration of the static and dynamic forces acting on a solid overflow diversion dam built across the river:

First, for weirs built on impervious foundations, such as bedrock not fissured, or impervious clays.

Second, for weirs on pervious foundations of silt, sand or gravel.

The forces acting on weirs built on impervious foundations include the following static forces and dynamic forces.

The static forces are:

1. The normal water pressure on the upstream face.
2. The normal water pressure on the downstream face due to the back water.
3. The weight of the water supported by the crest.

The dynamic forces are:

1. The erosive or scouring force on the downstream side of

the weir produced either by the high velocity or by the impact of the water pouring over the weir.

2. The force of impact of ice, trees, etc., against the crest on the upstream face of the weir.

The forces acting on weirs built on pervious foundation include in addition to those stated above the following underflow forces:

1. The transporting and erosive power due to the underflow in the stream bed under the weir floor.

2. The upward static pressure on the weir floor produced by the water under the floor.

Theoretical Cross Section of a Gravity Masonry or Concrete Weir on an Impervious Foundation.—A diversion weir differs from the usual dam used for storage purposes, in that water overflows the crest. The conditions for stability against the hydrostatic pressure are the same but require the consideration of the additional forces produced by the overflow water. A diversion weir is usually a low structure and therefore does not have to be designed for the crushing of the masonry, which is only to be considered in high dams usually over 100 feet in height. The conditions for stability against hydrostatic pressures are:

1. There must be no tension in the masonry or in the contact plane between the weir wall and the foundation.

2. There must be no overturning.

3. There must be no tendency to slide on the joint with the foundation or on any horizontal plane above the base.

4. The maximum pressure on any plane or on the foundation must not exceed the prescribed safe limit.

The simplest application of these conditions of stability is obtained from a consideration of the theoretical cross section required for a dam not subject to overflow, with the crest of the dam level with the surface of the water. The most economical theoretical cross section is triangular, with the upstream face vertical (Fig. 5A).

The first condition is met when the section is designed of such width that the resultant of the hydrostatic pressure with the weight of the masonry shall cut the base at or within the middle third. When there is no water back of the dam the resultant is the weight of the dam, and for a triangular profile with a vertical backface the resultant will fall exactly at the

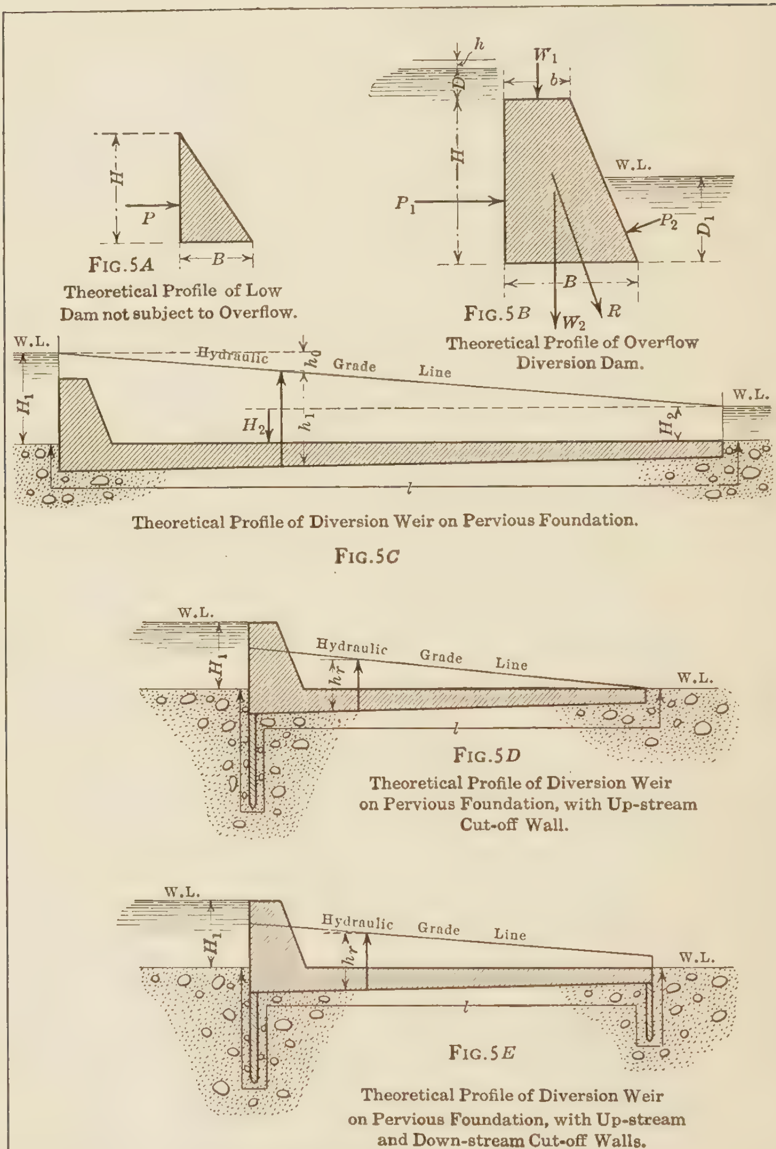


FIG. 5.—Theoretical profile of diversion weirs.

upstream end of the middle third. When there is full hydrostatic pressure up to the crest of the dam the resultant must fall not farther out than the downstream end of the middle third; this condition determines the base width, which is obtained as follows (Fig. 5*A*):

Assume a lineal foot of dam:

Let P = total full hydrostatic pressure.
H = height of dam.
d = density of concrete or masonry.
w = weight of a cubic foot of water.
B = base width.
W = weight of dam section.

$$P = \frac{wH^2}{2} \text{ and } W = \frac{BH}{2}dw.$$

$$\frac{wH^2}{2} \times \frac{H}{3} = \frac{BH}{2}dw \times \frac{B}{3}.$$

From which:

$$B = \frac{H}{\sqrt{d}}$$

Using 62.5 pounds and 150 pounds for the respective weights of a cubic foot of water and of concrete, the density is 2.4 and the base width is $B = 0.65H$.

This relation produces a minimum intensity of pressure at the upstream toe of the base of zero and a maximum intensity of pressure on the base at the downstream toe of the dam equal to $\frac{2W}{B}$, which is twice the average intensity of pressure on the foundation. It fulfills the second condition, giving a factor of safety against it of two. To meet the third requirement, the coefficient of friction must be at least equal to 0.65. The coefficient of friction of a masonry weir on a medium smooth rock foundation will be about equal to or greater than the above value. Additional strength against sliding is produced by the bond between the weir and the rock foundation by leaving the plane of contact rough and by forming it into steps or by cutting anchor trenches. The coefficient of friction when the weir is built on a clay sand or gravel foundation will be less than the required value given above. The required additional strength against sliding is obtained by anchorage to cut off walls, or piles, and by the resistance to friction obtained from the floor,

which for soft foundations is necessary and is built as part of the weir.

The design of the theoretical profile of an overflow masonry weir to resist the static forces requires the consideration of the following hydrostatic pressures: (1) The normal water pressure on the upstream face. (2) The normal water pressure on the downstream face. (3) The weight of the water on the crest of the dam. These pressures must, with the weight of the weir wall, give a resultant pressure which will fall within the middle third. These forces are indicated on the accompanying profile (Fig. 5B). Consider a lineal foot of weir wall and let:

b = width of crest of weir.
B = width of base of weir.
H = height of weir.
D = depth of overflow.
h = head due to velocity of approach.
D_1 = depth of backwater or tail-water.
P_1 = normal water pressure on the upstream face.
P_2 = normal water pressure on the downstream face.
W_1 = weight of water on weir crest.
W_2 = weight of weir wall.
d = density of concrete or masonry.

The approximate dimensions of the weir wall may be determined by assuming it as a simple triangular profile extending to the water surface, its height being equal to $H + D + h$. The crest width at the depth $D + h$ is then $b = \frac{D + h}{\sqrt{d}}$, and the base width is $B = \frac{H + D + h}{\sqrt{d}}$.

The crest width thus obtained is smaller than is generally used in practice. A greater width is necessary to give it added strength against the dynamic forces, such as the impact of ice and floating material. W. G. Bligh, a retired executive engineer of India, recommends the following formula for the crest width: $b = \sqrt{H} + \sqrt{D}$.

These dimensions are only approximate and the profile must be checked by computation either analytically or graphically and modified if necessary. The correct cross section is obtained when the maximum resultant pressure falls just at the downstream end of the middle third. The value of the resultant

pressure will vary with the relative elevations of the water surfaces on the upstream and downstream sides of the weir wall, corresponding to the variations in stream flow.

The depth of water above the weir crest is obtained by one of the standard formula for flow of water over weirs. Francis formulæ are generally used.

$Q = ClH^{3/2}$ for no end contractions, no submergence and no velocity of approach.

$Q = Cl\ [(H + h)^{3/2} - h^{3/2}]$ for no end contraction, no submergence and with velocity of approach.

$Q = Cl\ (NH)^{3/2}$ for no end contraction and with submergence.

In these formulæ:

Q = discharge over weir for length of weir crest equal to l.
C = coefficient varying with the form of the weir crest.
H = head on crest of weir.
h = velocity head
N = a coefficient which depends on the proportional submergence.

Values of C and N are given and referred to more in detail in the discussion of flow over weirs.

The elevation of the backwater or downstream water level will be obtained from the depth of water in the river channel, corresponding to the volume of water carried, by the formula for flow of water in channels: $Q = AC\sqrt{rs}$ where A = area of water cross section, r = hydraulic radius, s = grade of river, C = coefficient obtained by Kutter's formula. For a broad channel, the value of r will be nearly equal to the depth of water, and with the weir base on the same grade as the bed of the river, $D_1 = r$ (approximately).

The dam should be designed for the most severe conditions. This will not necessarily be either when the water level on the upstream side is level with the crest, with no overpour and therefore no backwater, or when there is maximum flood flow with greatest depth of water on the upstream and downstream side. The design should be considered for other conditions as well as these conditions, by using for each stage of flow the corresponding upstream and downstream elevations of the water surfaces. Where the downstream water level may rise above the crest of the dam so as to submerge it, the most severe condition will occur when the weir is near the point of submergence.

Upward Pressure Due to Infiltration or Percolation under the Weir Wall.—When a weir is built on fissured rock or on a solid foundation in such a way that the contact between the base of the dam and the foundation is not perfect and allows percolation under the dam, the water, if not removed by drainage, produces an uplift pressure on the base, which must be considered in the design of the cross section. The intensity of this upward pressure will vary with the distance the water has to percolate and will therefore depend on the total perimeter of contact between the base of the dam and the foundation, including the cut-off trenches. The intensity of pressures at the entrance and at the outlet of the path of percolation between the two surfaces are equal respectively to the depth of water at the upstream and downstream toe of the weir. The total uplift pressure will vary with the stages of flow; the values obtained for different conditions of flow must be considered with the corresponding static pressures on the upstream and downstream faces and on the crest of the dam, in order to obtain the most severe conditions on which the design must be based.

In order to prevent or decrease the uplift pressure, the base may be drained, by placing a line of drains under the base parallel to and near the upstream toe and discharging the collected water by cross drains (Fig. 12*B*). The maximum uplift pressure will then correspond to the maximum level of the backwater.

Theoretical Cross Section of a Gravity Masonry or Concrete Weir and of Loose Rock Weirs on a Pervious Foundation.—The design and construction of diversion weirs on pervious foundations has usually been largely based on the design of existing structures of the same type and under similar conditions, modified according to the judgment and experience of the engineer, using more or less empirical rules. In a text-book on the Practical Design of Irrigation Works, the author, W. G. Bligh, a retired executive engineer of the Indian Public Work Department, has evolved principles of design for such works, based largely on the dimensions of structures in India, and on experiments on the passage of water through sand by Lieut.-Col. J. Clibborn of Roorkee, India.

Types of Weirs Used in India.—The diversion weirs built in India on pervious foundations are of three types: (1) Those which consist essentially of a weir wall of gravity section, usually trapezoidal, and of a strong floor or apron on which the water

falls (Figs. 5*C*, 5*D*, 5*E*). (2) Those which consist of a raised sill across the river with an upstream impervious floor, usually of masonry, extending from the stream bed and sloping up to the crest of the sill, and a downstream impervious floor of masonry sloping down from the crest of the sill to the stream bed and continued with riprap or paving. The crest of the wall is often equipped with collapsible gates or shutters (Fig. 6). (3) Those which have a flat triangular profile similar to the second class, but built of loose rock fills between longitudinal, rectangular, cross walls parallel with the axis of the dam, one of which is placed on the crest line and forms the crest. In this type the upstream face of the rock fills slopes up from the stream bed to the crest on a slope of usually 1 on 3, or 1 on 4, and the downstream or rollerway face slopes down from the crest to the stream bed on a slope of from 1 on 10, to 1 on 15, depending on the character of the stream

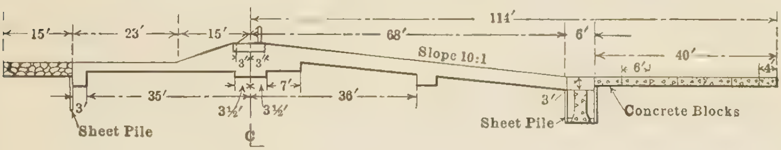


FIG. 6.—Jamrao Weir, India.

bed. The Laguna weir on the Yuma Project, California-Arizona (Fig. 8), described farther, is of this type, differing from the Indian weir type, in that the walls are extended down below the stream bed and sheet piling is used below the crest wall. In India the walls extend only down to the stream bed. This class of weir differs from the first and second class in that the body of the weir is pervious and therefore not subject to hydrostatic uplift pressure. The first type of weir, often modified by shaping the downstream face of the weir to an Ogee form, as described farther, is that most commonly used in the United States for solid masonry or concrete weirs. For this reason the principles of design stated below have been applied specially to this type, although they are equally applicable to the second type and in part to the third type.

Design of Solid Masonry or Concrete Weir on Pervious Foundation.—The weir will consist of the superstructure or weir wall across the river, the downstream impervious floor or apron, with an extension of riprap or paving on the stream bed, and usually a cut-off wall or sheet piling at the upstream toe of the

weir wall and one at the downstream end of the floor (Fig. 5*E*). In some cases an upstream floor is used in addition to the upstream cut-off wall, or as a substitute for it.

Forces Acting on the Weir.—The forces include, in addition to those acting on a weir built on an impervious foundation, the hydrostatic uplift pressure and the underflow transportive or erosive power. The weir wall above the floor may be considered as built on the impervious floor and is then designed in the same manner as if built on an impervious foundation according to the principles presented above. The design of the other parts of the weir is determined by the hydrostatic uplift and the underflow erosive force which depends on the velocity of the undercurrent and is proportional to the difference in water levels on the upstream and downstream sides and inversely proportional to the length of the path of percolation. The important principles of design of weirs on pervious foundations are:

First.—The path of percolation under the weir must be made large enough to offer sufficient resistance to the underflow to prevent the undermining of the structure by a too high underflow velocity.

Second.—The floor must be designed of sufficient thickness to resist the uplift pressure.

Hydrostatic Uplift Pressure.—The hydrostatic uplift pressure acting on the underside of the weir base or floor will depend on the elevations of the water surfaces on the upstream and downstream sides of the weir. When the water level on the downstream side is higher than the surface of the floor, so as to submerge it, then the uplift pressure is partly balanced by the weight of water on the floor (Fig. 5*C*). The resultant unbalanced upward water pressure will therefore be maximum when the difference in elevation between the water surfaces is maximum. This will usually be obtained when the stream flow is minimum or when the water level on the upstream side is level with the crest of the weir and when the stream below is either dry or when the downstream water level is just level with the floor. An increase in flow will produce a depth of overpour over the dam and a rise in the backwater level. On streams with sandy channels, the grade is flat and the velocity is low. For these conditions the computations will usually show that the depth of overpour will be smaller than the corresponding rise in the

backwater level, so that the difference between the water levels on the upstream and downstream sides of the weir will decrease for an increase in flow. The intensity of uplift pressure is maximum at the upper or inlet end of the path of percolation; from this point it decreases in proportion with the distance the water has to creep.

Path of Percolation.—Mr. Bligh states that, from experiments made in India, piezometer measurements have shown that in considering the uplift pressure the path of percolation is not the shortest line between the inlet and outlet, but is the path formed by the perimeter of contact between the material of the stream bed and the under side of the weir with its floor, cut-off walls, sheet piling and other projections and that when water-tight cut-off walls or sheet piling are used, both at the upper end and lower end of the floor and placed not closer to each other than twice the maximum depth, the path of percolation will be down the upstream face of the wall and up along its downstream face, then along the plane of contact of the weir wall and floor on the porous material, and finally down and up the second cut-off wall (Fig. 5*R*). The effect, therefore, of using cut-off walls is to increase the length of the path of percolation by twice the depth of each cut-off wall. The upper cut-off wall at the upstream toe of the weir decreases the uplift pressure on the floor, but the lower cut-off wall at the downstream end of the floor increases the intensity of uplift pressure. The same effect as obtained with the upper cut-off wall is produced by the use of an impervious floor constructed on the bed of the stream channel on the upstream side of the weir, equal in length to twice the depth of the sheet piling. This upstream floor may be made thin because the depth of water on the floor will more than balance the uplift pressure on the under side of the floor. The main purpose of the lower cut-off wall is to either give a sufficient length of enforced percolation or to protect the downstream end of the floor from undermining by the backwash of the water. If a sufficient length of enforced percolation is obtained without the path around the lower cut-off wall, drain holes through the wall or at the lower end of the weir floor are advisable to decrease the uplift.

Length of Path of Percolation.—The path of percolation must be great enough to offer sufficient resistance to the underflow to prevent the undermining of the structure by the transportation

or washing away of the material under the floor. The necessary length of enforced percolation has been determined largely from observation of existing structures and from failures due to the underflow. Mr. Bligh recommends the following relation:

$$l = C \times H$$

where l = length of path of enforced percolation in feet.

H = head of water in feet represented by the maximum difference in elevation of the water levels on the upstream and downstream side of the weir.

C = coefficient depending on the character of the material of the stream bed.

C = 18 for river beds of light silt and sand as the Nile.

C = 15 for fine micaceous sand such as in the Colorado River.

C = 12 for coarse-grained sand.

C = 9 for gravel and sand.

C = 6–4 for boulders, gravel and sand mixed.

Thickness and Weight of Floor.—The intensity of the uplift pressure at any point is, as stated above, dependent on the relative elevation of the water surfaces on the upstream and downstream sides of the weir, and on the position of the point considered on the path of percolation. When the floor is submerged the uplift pressure is partly balanced by the weight of water on the floor at that point and the resultant represented by the difference is the upward pressure which the floor must resist. This resistance may be obtained:

First.—By connecting the floor to anchor piles or other form of anchorage.

Second.—By making the floor sufficiently thick to give a weight in excess of the upward pressure.

The inlet and the outlet to the path of percolation are usually at the stream bed, and the corresponding pressure heads at the inlet and outlet are the depths of water above the stream bed measured respectively on the upstream and downstream sides of the weir. The intensity of uplift pressure at any point on the underside of the floor is best obtained by a graphical study of the weir cross section and hydraulic pressures. The hydraulic grade line must be drawn, and the distance measured from the hydraulic grade line to the underside of the floor at the point considered gives the uplift pressure head. The hy-

draulic grade line is broken and dropped at vertical cut-off walls a distance equal to the loss in head produced by this part of the path of percolation, and between cut-off walls or other projections below the floor it will have a uniform slope unless the material of the stream bed along the path of percolation changes or the base of the floor forms more than one continuous plane. To obtain the intensity of pressure at any point for the general case (Figs. 5*C*, 5*D*, 5*E*).

Let H_1 = pressure head at the inlet to the path of percolation, equal to the depth of water on the upstream side of the weir.

H_2 = pressure head at the outlet to the path of percolation, equal to the depth of tail-water above the downstream end of the floor.

l = total length of the path of percolation.

l_1 = length of path of percolation to the point considered.

$h = H_1 - H_2$ = total drop in hydraulic grade line or loss in pressure head or in uplift pressure in the distance l.

h_0 = drop in hydraulic grade line or loss in pressure head corresponding to the length of the path of percolation down to the point considered.

h_1 = pressure head at point considered, measured from the hydraulic grade line.

p_1 = intensity of uplift pressure at the point considered in pounds per square foot.

h_2 = depth of water above floor at point considered.

p_r = resultant intensity of upward pressure at the point considered to be balanced by the weight of the floor in pounds per square foot.

w_1 = weight of a cubic foot of water in pounds.

w_2 = weight of a cubic foot of floor material in pounds.

t = thickness of the floor in feet.

$$h_0 = \frac{h}{l} \times l_1.$$

$$p_1 = h_1 \times w_1.$$

$$p_r = (h_1 - h_2)w_1.$$

In determining the required thickness of the floor, the conditions giving maximum value for p_r must be considered. As previously stated, these conditions are when there is maximum

difference in elevation of the water surfaces, which will usually be when the water level on the upstream side of the weir is level with the crest of the weir and the water level on the downstream side is minimum or level with the lower end of the floor. For these conditions giving maximum resultant upward pressure and with a level floor surface, h_2 will be zero, and if the special value of h_1 is denoted by h_r, then $p_r = h_r w_1$.

Where the floor is designed to resist upward pressure by anchorage, the floor is divided by the anchorage into panels which must be designed as floor slabs against the bending stresses produced by the upward pressures determined as explained above. Where the floor has no anchorage and bending strength, such as when built of masonry or plain concrete not anchored, the weight alone prevents the uplift. To give ample safety, the thickness of the floor may then be made ⅓ greater than that required for exact balance. The thickness is then $t = \frac{4}{3} \frac{h_r w_1}{w_2}$.

Length of Floor and Cut-off Walls.—The total length of the path of percolation is obtained as previously stated. This length is divided up between the downstream floor, the cut-off walls, and in some cases an upstream floor. The downstream floor may be made of such length that it will comprise a large part of the path of percolation, but it must not be made too short, for one of the main objects of the floor is to protect the stream bed against the impact of the falling water. For this purpose a length of floor, measured from the downstream toe of the weir wall, of 3 or 4 times the height of the weir crest above the floor is sufficient. Mr. Bligh recommends the following empirical equation for the length of apron or floor measured from the downstream toe of the weir wall:

$$L_a = 4C\sqrt{\frac{H_a}{13}}$$

where C = coefficient used to determine the length of the path of percolation.

H_a = height of weir crest above the floor or apron.

Knowing the length of the floor, this value subtracted from the total path of percolation gives the length to be divided between the cut-off walls or to be given to an upstream floor.

Riprap Extension for River-bed Protection.—The impervious floor is continued downstream with riprap or paving. The

length of this riprap must depend on the character of the stream bed, the height of fall and the volume of water carried. The total length of floor and riprap is stated in the form of an empirical equation given by Mr. Bligh:

$$L = 10C\sqrt{\frac{H_b}{10}}\sqrt{\frac{q}{75}} = 0.355C\sqrt{H_b q}$$

Where L = total length of floor and riprap or paving.
H_b = the height of the weir crest above low water level on the downstream side of the weir wall.
q = maximum flood discharge in cubic feet per second per lineal foot of weir crest.
C = path of percolation coefficient (page 24).

The length of riprap or paving alone will be:

$$L_p = L - L_a = 0.355C\sqrt{H_b q} - L_a$$

The thickness of loose riprap should be not less than 2 feet.

DESIGN OF DIVERSION WEIRS OF THE LOOSE ROCK-FILL INDIAN TYPE

This type of weir, previously described, is illustrated by the Okhla weir (Fig. 7) on the Jumna River, India, and the Laguna

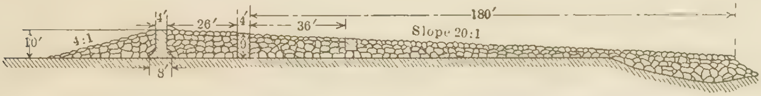


Fig. 7.—Okhla weir, Jumna River, India.

weir (Fig. 8) across the Colorado River (Arizona-California). This type of weir has a flat triangular profile, built of loose rock fills separated longitudinally by rectangular vertical walls or core walls parallel with the axis of the weir. The main wall is placed on the axis of the crest of the weir. On the upstream side of the main crest wall the loose rock fill slopes from the crest down to the stream bed on a slope ranging usually from 2 to 1, to 4 to 1; on the downstream side the rock fill has a flat slope, usually from 12 to 1, to 20 to 1, and is divided by one or more walls parallel with the crest wall. The division walls are usually 4 to 6 feet thick; their bases rest on the stream bed or may extend a short distance below the stream bed. The walls are spaced from 30

to 45 feet apart, which gives a drop in elevation in the crest of the walls from one wall to the next downstream one of 2 to 3 feet.

The principles of design are not as definite as for the type of Indian weir, which has a solid impervious floor. The rock fill on the upstream side will in time become more or less impermeable by the deposition, in the pore space, of silt and sand carried by the water, and will then form an upstream apron which will increase the resistance to percolation through it and will add to the length of the path of percolation. The downstream rock fill will become less permeable by the silt and sand deposited by the river water, and also by some of the finer material of the stream bed washed up by the upward current of the underflow water.

As the impermeability of the upstream rock fill cannot be depended upon, the length of the downstream rock fill is usually made sufficient to give the required path of percolation, as with the weirs built with an impervious floor. This length will, therefore, be $C \times H$ where C is the coefficient previously given, ranging from 4 to 18, depending on the material of the stream bed, and H is the maximum difference in elevation of the water levels on the upstream and downstream sides of the weir, often equal to the height of the weir crest above the stream bed.

The uplift pressure considered in the design of impermeable floor is not a factor in the design of the rockfill type; the action which must be considered is the underflow current, which occurs by the water passing from one basin under the base of the dividing wall into the next lower basin. The water level in each basin will rise to the crest of the downstream wall of the basin so that the underflow current is produced by the difference in adjacent water levels. The velocity of this current is dependent on this difference in water levels and on the resistance offered by the path of percolation, which extends down through the rock fill of the upper basin, then under the base of the dividing wall and up through the rock fill of the adjacent basin. Mr. Bligh in his criticism of the Laguna weir states that the sheet piling forming the extension below the base of the crest wall appears unnecessary, as he believes it is sufficient to have the base of the walls built directly on the bed of the channel as in the case of the Okhla weir, with no extension into the stream bed. But the writer believes that although it may not be necessary, the sheet piling or extension is desirable, for it increases the path of percolation around the main crest wall from the upstream side into

the first or upper basin and produces additional safety at a relatively small cost.

To conform with the principles of underflow previously stated, it would seem that the path of percolation from one basin into another should be sufficiently long to produce a resistance which will prevent a velocity of such magnitude that some of the material of the stream bed be washed up into the rock fill and produce settlement. The resistance to flow in the loose rock fill is small, at least when first constructed before finer material, such as sand and silt, has been deposited in the voids; therefore the resistance must be largely obtained from that part of the path of percolation in the sand around the base of the wall. To increase this path it is desirable to extend the base of the wall below stream bed. Neglecting the resistance to flow offered by the rock fill, the path of percolation in the stream channel around the base of the wall need not exceed CH_1 where C is the coefficient previously given for different classes of material and H_1 is the difference in elevation between water levels on both sides of the wall. If this path of percolation is not of sufficient length, there will be a gradual adjustment of the material in the stream bed, with more or less settlement. In some of the older weirs built in India the usefulness of more than one cross wall along the axis of the crest was not realized, so that the use of a single cross wall produced an excessive velocity around the base of the cross wall which washed up considerable material from the stream bed and allowed at the same time settlement of the loose rock until a natural mixture of sand and rock offering sufficient resistance to the flow was obtained. After settlement the loose rock fill was brought up to its required height by additional material.

To protect the downstream loose rock fill against the high velocity, the sloping face is paved with larger rock; this paving or riprap should extend beyond the toe of the slope to protect the stream bed down to a point where the velocity will be decreased to its normal value. The total length from the crest of the weir to the end of this protection is obtained by the same formula as previously given for weirs with impervious floors.

Laguna Diversion Weir on Colorado River, Yuma Project (Arizona-California)—(Figs. 8 and 9).—This weir is built on the lower part of the Colorado River where the river bed is sandy material down to a great depth. The stream flow is subject to great variations, ranging from a low water flow of 3,500 to 4,000 second-feet

up to a maximum flood flow of about 150,000 second-feet (1909 and 1912). The water carries large quantities of silt. The most favorable site for the construction of the weir was about 12 miles above Yuma, where the width of the flood-flow channel, between the granite outcroppings, which form the banks of the river, is nearly 1 mile. Above this point, the grade of the stream bed is about 1 foot per mile, and the stream channel is continuously changing its banks. To best meet the conditions, the type of weir selected was the Indian type of rock-fill weir, used successfully in India and Egypt. The weir is 4,800 feet long, and raises the water level of the river about 10 feet to divert water into a main canal of 1,400 second-feet capacity on the west bank of the river in California and into a smaller canal on the east bank of the river in Arizona (Fig. 9). One of the most interesting fea-

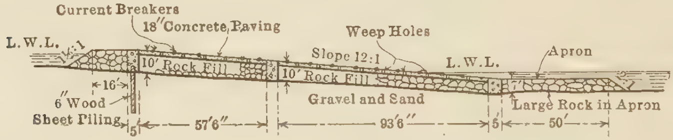


FIG. 8.—Laguna weir.

tures of the diversion works is the type of sluiceway channel and of canal headgates at each end of the weir, designed specially to carry the minimum amount of sediment in the canals; these parts of the diversion works are described in the discussion of scouring sluices.

The diversion weir is formed of three parallel walls across the river, with rock filling between these walls and against the upstream face of the crest wall, a concrete pavement 18 inches thick on the rock fill between cross walls, and an apron of large derrick-size rock extending for 50 feet downstream from the toe wall. The space between the crest wall and the middle wall is 57 feet, and between the middle wall and toe wall 93 feet. The base of the cross walls rests on the stream bed, and at the base of the crest wall a cut-off wall of wooden sheet piling 6 inches thick extends from 12 to 20 feet in the stream bed. The height of the crest wall above the stream bed is 19 feet near the deepest part of the main channel and decreases toward the end.

The concrete pavement is built in slabs about 10 feet wide and 15 feet long, with open joints to relieve upward pressure. The downstream apron is 7 feet thick with its upper surface

about 3 feet lower than the low water level in the river. To prevent parallel currents along the downstream end of the apron, which would be produced by the water passing over the weir at

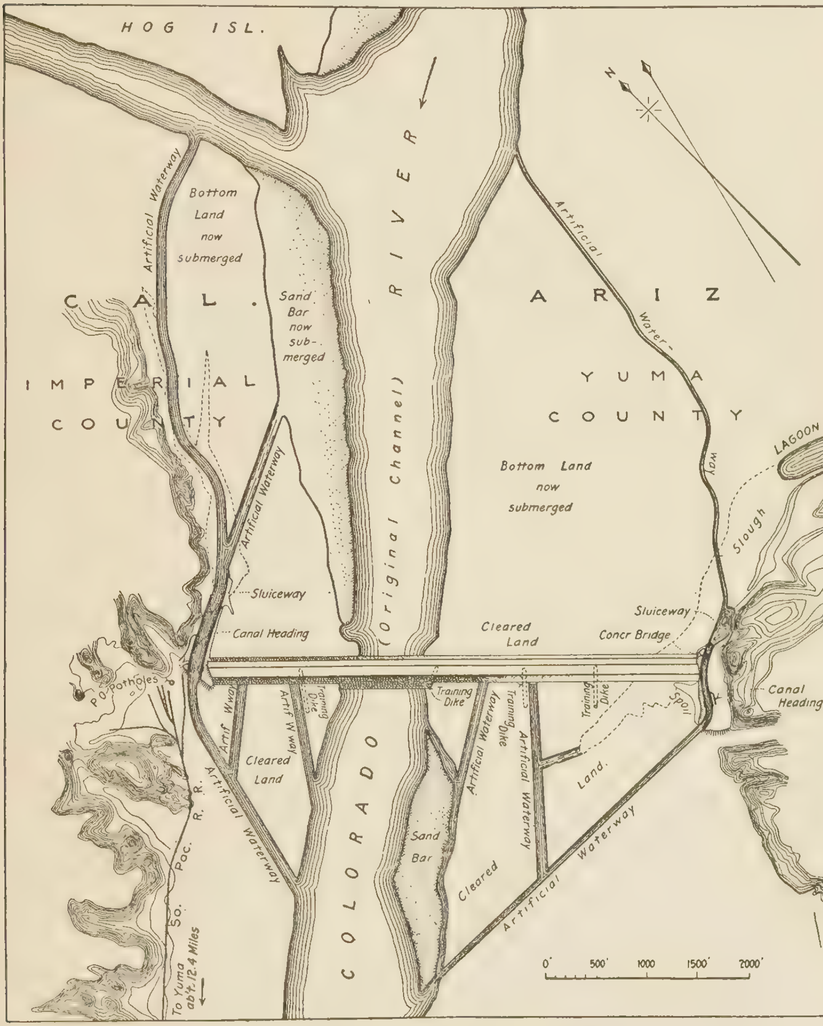


FIG. 9.—General plan of Laguna dam, sluiceways and canal headings.

the two ends and moving toward the deeper portion of the stream bed, four training dikes were built and four artificial waterways were excavated from the end of the apron (Fig. 9). The main

feature of the construction and the great difficulties which had to be overcome to complete the weir in the spring of 1909, involving the closure of the deepest part of the stream, left open till the end, are well described by E. D. Vincent in an article given in the list of references.

DYNAMIC FORCES PRODUCED BY FLOW OF WATER OVER WEIRS AND THEIR EFFECT ON THE DESIGN OF WEIRS

The following dynamic forces are obtained:

First.—An erosive or scouring force on the downstream side of the crest resulting from the high velocity or the impact, or both, of the water pouring over the crest.

Second.—The force of impact of floating ice, trees, etc., on the upstream face of the weir.

The first force will act on the downstream face of the dam and also on the stream bed below. To resist it, the following types of construction have been used, either singly or in combination:

1. An Ogee shape for the downstream face of the weir to prevent impact.
2. A flat rollerway or sloping downstream face of the weir.
3. A series of steps for the downstream face of the weir.
4. A strong apron which will resist the impact and destroy the velocity.
5. A water cushion in which the water falls.
6. An apron of riprap, paving or lining, built as an extension beyond the downstream end of the weir to protect the stream bed.

The first two forms of construction prevent impact but do not destroy the velocity.

To determine the principles of design of these different forms of construction, it is important to first consider the principles of flow of water over weirs.

Flow of Water over Weir Crests.—The water discharged over a weir crest takes the path of a falling body moving with an initial horizontal velocity. This curve, which is a parabola, has the following equation:

$$y = \frac{g}{2v^2} x^2$$

where:

v = initial horizontal velocity of the overpour water at the weir crest.

y = ordinate corresponding to abscissæ x both measured from the origin of the curve of falling water.

To determine the velocity (v) it is necessary to know the discharge and the exact water cross section (A) directly at the weir crest; when these are known, $v = \frac{Q}{A}\cdot$ As the discharge Q and the length of weir crest are known for the case considered, the only remaining factor to be known is the depth of water at the weir crest. The depth of water obtained with the weir formula, as generally used, is the difference in elevation between the weir crest and the water level measured at a point some distance upstream from the weir crest, before the water level begins to drop. From this point down the water level drops on a curve, so that the depth of water on the crest (D) is considerably less than the head of water (H) used in the weir formulæ.

The weir formulæ most generally used are the following:

East Indian engineers' formula:

$$Q = CLH^{3/2} \text{ where } C = 3.4989 - 0.0535H$$

This formula provides no correction for velocity of approach.

Bazin formula:

$$Q = \left(0.405 + \frac{0.0984}{H}\right)\left(1 + 0.55\,\frac{H^2}{(P+H)^2}\right)LH\sqrt{2gH}$$

This formula considers velocity of approach.

Francis formula:

$$Q = CLH^{3/2} \text{ (no velocity of approach).}$$
$$Q = CL[(H+h)^{3/2} - h^{3/2}] \text{ (with velocity of approach).}$$

In the formulæ the following notation has been used:

P = height of weir in feet (used in Bazin's formula only).

H = measured head in feet or difference in elevation between weir crest and water level, at a point sufficiently far upstream to avoid the surface curve. A distance from the weir crest to the point of measurement equal to 2½ times the height of the crest of the weir above the bottom of the channel is recommended by Boileau.

h = head in feet corresponding to velocity of approach.

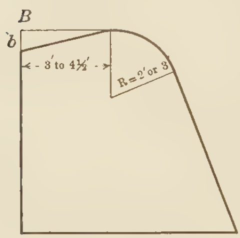
L = length of weir crest in feet.
C = coefficient of discharge.

When the water level on the downstream side is above the crest of the weir, the weir is submerged. For these conditions Clemens Hershel's formula for a thin edge or sharp edge weir is:

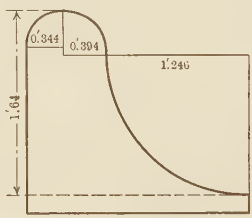
$$Q = 3.33\,L(NH)^{3/2}$$

Where N is a coefficient which depends on the proportional submergence, a few values of which are as follows:

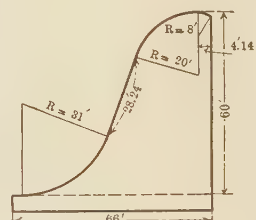
Ratio of submergence in per cent.........	10	20	30	40	50	60	70	80	90	99
Values of N..........	1.005	0.985	0.959	0.929	0.892	0.846	0.787	0.703	0.574	0.275



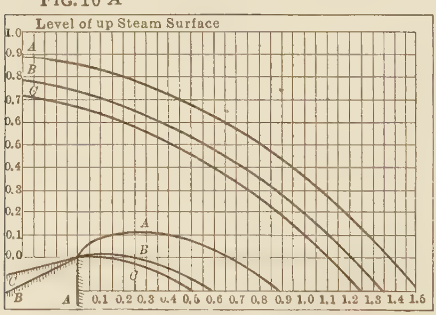
Typical Weir Crest for which Coefficient Formula is Derived
FIG. 10 *A*



Bazin's Model of Curved Crest Dam
FIG. 10 *C*



Austin Dam, Texas
FIG. 10 *B*



Bazin Weirs
Profile of Upper and Under Surface of Sheet of Falling Water for Varying Inclinations of Approach to Weir Crest, According to Bazin's Experiments
FIG. 10 *D*

FIG. 10*D*.

This formula may be applied to broad crested weirs by substituting for the coefficient 3.33, special values depending on the shape of the crest, such as are given below for free discharge coefficients. If the formula be corrected for velocity of approach it may be written as a modified form of Francis formula:

$$Q = CLN^{3/2}[(H + h)^{3/2} - h^{3/2}]$$

Francis formula is the one most generally used in the United States. In this formula the value of the coefficient, which

depends on the form of weir crest and depth of water, varies from about 2.5 to 4.5 and mostly between 3.0 and 4.0. The following formula has been deduced by the U. S. Geological Survey for the coefficient for weirs of Ogee section, with 2 or 3 feet crest radius and upstream slopes 3 to 4.5 feet broad (Fig. 10A). (Page 131, Water Supply and Irrigation Paper No. 200, U. S. Geological Survey.)

$$C = [3.62 - 0.16(S - 1)]H^{1/20}$$

Where S = batter ratio of the slope, $\frac{\text{horizontal run}}{\text{vertical rise}} = \frac{B}{b}$.

Measurements of discharge by the U. S. Geological Survey over an actual Ogee dam (Austin dam, Texas) (Fig. 10B) gave the following results:

Discharge Coefficients for the Austin, Texas, Dam

H	D = depth at crest	Ratio $\frac{D}{H}$	Average value of C
0.42	0.33	0.79	3.112
0.72	0.625	0.87	3.053
1.09	0.838	0.77	3.132
1.32	0.96	0.727	3.302
1.45	1.04	0.717	3.333

With the above formulæ and values of discharge coefficient, the value of H is obtained for any corresponding discharge. The value of D (depth of water directly on crest of weir) is required to find the velocity at the crest which determines the equation of the parabola. The value of D as obtained by the measurements on the Austin dam, expressed in percentage of the depth H, ranges from 0.87 for small depths of water to 0.717 for the larger depth of water. Measurements on curved crests of dams of similar form made by Bazin on small models (Fig. 10C) give the following results for one of the most comparable models:

Head, H	Depth on crest, D	$\frac{D}{H}$
0.305	0.239	0.78
0.574	0.433	0.75
0.820	0.604	0.74
1.089	0.820	0.75

On other models of curved crests the results, obtained by Bazin, are very nearly the same (Annales des Ponts et Chaussees, Vol. II, 1898, page 151–264). Other measurements of profiles of

the sheet of falling water are reported on pages 111, 112 of the U. S. Geological Survey Water Supply Paper No. 200. On flat crests, weirs 6.56 and 6.89 feet wide and for maximum heads of water of 5.15 and 4.65 feet respectively, the ratio of $\frac{D}{H}$, where D is measured at the center of the crest, was 0.61 for both weirs, and where D is measured at the downstream edge of the crest, the ratio was 0.43 and 0.45 respectively. For a flat-crest dam 8.42 feet wide (Cleggs dam, North Carolina) Elwood Morris obtained for a head of 1.25 feet the ratio 0.40.

As it is important, in order to prevent a vacuum under the sheet of falling water, to determine the profile of this sheet of water, for maximum flow condition, when the curve is the flattest, it will be best to assume that the depth of water on the crest of an Ogee dam properly curved will be 65 per cent. of the head H.

From the equations given above, we then have for the velocity at the crest:

$$v = \frac{Q}{A} = \frac{CLH^{3/2}}{L\frac{65}{100}H} = \frac{100}{65}CH^{1/2}$$

and,

$$y = \frac{g}{2v^2}x^2 = \frac{0.21g}{C^2H}x^2$$

or approximately,

$$y = \frac{6.8}{C^2H}x^2 \qquad x^2 = \frac{C^2H}{6.8}y$$

This equation for the parabolic curve is based on the average velocity of the water cross section at the crest of the weir. Assuming that the thread of mean velocity is $\frac{6}{10}$ below the surface, the origin of the curve will be at a point directly above the crest at a height equal to $\frac{4}{10}$ of the depth of water at the crest. This curve, when plotted, will represent the thread of mean velocity of the falling sheet of water. The upper-surface curve and under-surface curve which form the boundary of the water sheet will be obtained by finding the thickness of the sheet at any point and plotting the points of the upper-surface and under-surface curve, at right angles to the mean velocity curve, at distances equal to $\frac{6}{10}$ and $\frac{4}{10}$, respectively, of the thickness of the sheet (Fig. 11). The thickness of the sheet at any point is found from the discharge per lineal foot of crest divided by the velocity at that point which is the resultant of the constant initial horizontal

velocity and of the vertical velocity due to gravity (equal to $\sqrt{2gy}$, where y = vertical distance below origin).

Ogee-shaped Downstream Face.—The object of forming the downstream face of an overflow dam or weir to an Ogee shape is to prevent the impact of the falling water on the stream bed or floor. The upper part of the face, from the crest down to a certain point, is formed to fit approximately the curve of the under face of the falling water; below this point the face is continued usually with a tangent connecting to a reverse curve, which changes gradually the direction of flow so as to discharge the water at the bottom fall in a horizontal direction parallel to the stream bed. In order to prevent the possibility of the formation

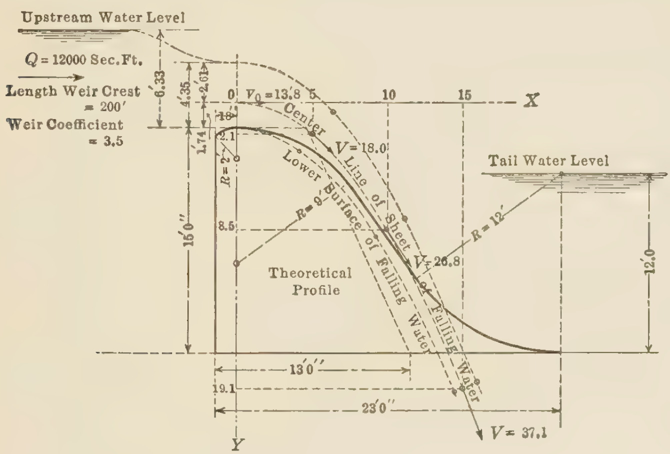


FIG. 11.

of a vacuum between the sheet of water and the face of the dam, it is safer to flatten the profile of the dam so that the downstream face will extend well into the sheet of falling water. The theoretical trapezoidal profile will first be determined and the downstream face is then modified to an Ogee curve (Fig. 11).

Where the vertical upstream face of the weir forms a right angle with the crest, there is full contraction of the under surface of the sheet of water, which causes, according to Bazin's experiments, a maximum rise in the surface equal to about $\frac{1}{8}$ of the head of water at a distance of about $\frac{1}{4}$ of the head (Fig. 10*D*). This contraction is nearly eliminated by making a battered approach to the crest of the weir of 1 vertical to 4 horizontal.

To eliminate this contraction, which will cause a vacuum, it is desirable to shape the crest of the dam with such a battered slope between the upstream vertical face of the dam and the summit of the weir crest, or to use a suitable curve which will fit this contraction. The profiles of the upper and under surface of a sheet of falling water, passing over a sharp edge crest, for a vertical face and also for downstream inclinations of 2 horizontal to 1 vertical, and 4 horizontal to 1 vertical, as obtained by Bazin are shown in the accompanying diagram (Fig. 10*D*).

The shaping of the downstream face of the weir to an Ogee face will change to a small extent the magnitude and position of the resultant weight of the weir section, which it is desirable to consider in a final study of the stability of the structure. The weight of the sheet of water on the Ogee face below the crest and above the tail-water will be comparatively small; it will add to the stability and may be neglected in the design.

To facilitate the construction of the downstream face, a circular arc is generally used to fit the curve of falling water and is continued by a sloping tangent to the curve which produces the deflection. The proper selection of the radius of the lower curve will depend largely on the judgment of the engineer. A study of a number of Ogee profiles of diversion weirs 10 to 30 feet in height show the following approximate relations:

First.—Radius of upstream curve at crest of weir = $\frac{1}{8}$ of the height of the diversion weir, or battered 1 foot vertically to 4 feet horizontally.

Second.—Radius of downstream curve at crest of weir made to fit curve of falling water and dependent on the depth of overpour water.

Third.—Radius of lower deflection curve = $\frac{1}{2}$ to $\frac{2}{3}$ of the height of diversion weir.

The use of Ogee faces has been quite commonly adopted for solid masonry and concrete diversion weirs, built for irrigation systems and hydroelectric power development projects in the United States (Fig. 12). The result of an Ogee face is the elimination of impact, but the water is discharged at the foot of the Ogee with a high velocity which, with stream channels formed of soft material, requires the protection of the stream bed for considerable distance downstream. According to a number of authorities on engineering practice in India, the use of Ogee falls in India has been abandoned and replaced by direct

falls on a strong apron for the reason that the action of the high scouring velocity resulting from the Ogee was more difficult to protect against than the force of impact. However, it is probable that the diversion weirs in India are largely built on

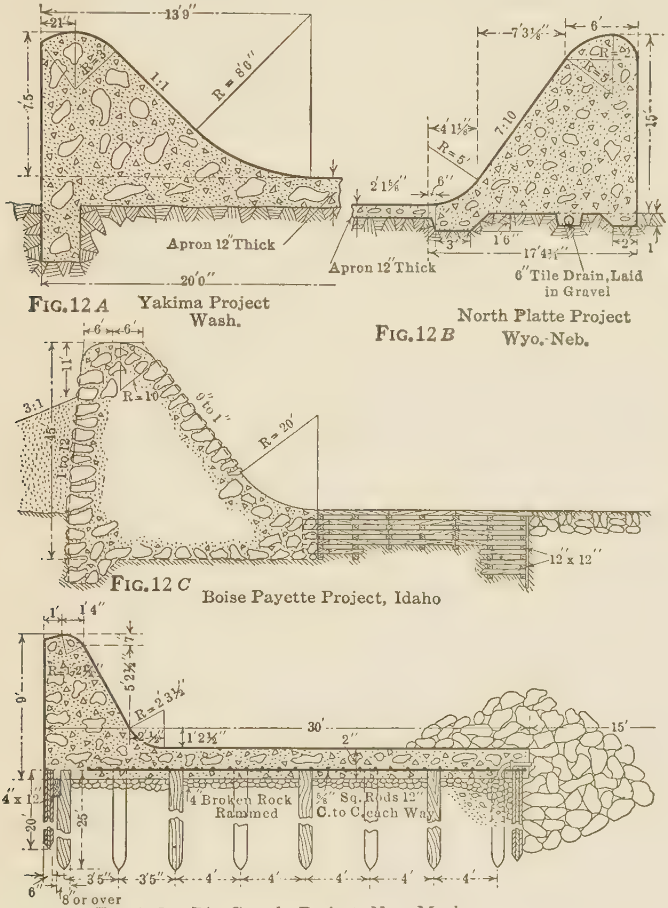


Fig. 12D. Rio Grande Project, New Mexico.

Fig. 12. Ogee Diversion Weirs.

soft foundations of silt, sand, or gravel and that the use of Ogee weirs is not as objectionable for more stable foundations.

Rollerway Downstream Face.—The term rollerway is used to designate the downstream face of a weir built on a flat slope, usually 12 to 18 feet horizontally to 1 vertically. The rollerway is intended to prevent impact and the surface is usually

made rough to offer greater resistance against the accelerated flow down the slope. This form of construction has been largely limited to diversion weirs of the loose rock type as built in India, an example of which is found in the United States in the Laguna weir on the Colorado River. The crude rock or cobble low diversion weirs built on some western streams are sometimes formed with a flat sloping downstream face somewhat similar to the rollerway of the Indian weir type, and in some cases the weir is given more permanency against flood flows by covering the loose rock fill with a layer of rock laid in cement mortar. The only other example of rollerway faces is found on a few timber dams, in which case the slope is steeper than that given above. The rollerway face is open to the same objection as the Ogee face, but to a less extent because the scouring velocity obtained at the foot of the rollerway is not as great as with the Ogee.

Stepped Downstream Face.—This form of downstream face is intended to destroy the impact force by dividing the total fall into a number of smaller falls and to prevent a high velocity at the foot of the last step. To be effective the steps must be made sufficiently long to receive the sheet of falling water inside of the upstream half of the step; otherwise the effect of the steps in destroying the impact will be greatly diminished, especially during heavy floods. This form is adopted with some form of crib weirs, but is expensive in amount of material and has little or no advantages over a direct fall on a heavy floor.

Downstream Floor or Apron to Resist Impact and Destroy Velocity of Fall.—This part of a diversion weir is used to protect the stream bed when there is a direct fall from the crest of the weir, and also as an extension to the toe of an Ogee fall.

When the foundation is impervious, hard, solid rock, the floor may not be necessary. On softer impervious rock, such as stratified rock, conglomerate rock and shales, a short floor at least is necessary. The floor length must be at least sufficient to receive the falling water well within the upstream half of the floor; this distance is obtained by considering the equation of the curve of falling water, but should not be less than 2 or 3 times the height of fall. For an Ogee face about the same length of floor should be used. The floor is usually made of concrete, masonry, framed timber, or timber cribs filled with rock. A concrete floor should not be less than 1 foot thick for

low falls, and preferably 2 feet thick for falls above 20 feet in height. The floor may have to be extended with riprap or paving.

When the foundation is pervious or soft material, such as fissured rock, sand, gravel, the floor is a more important part of the weir. The length and thickness of the floor depend on the character of the material, the height of the weir, the volume of water passing over the weir and the upward hydrostatic pressure under the floor, and is determined by the principles previously considered in the design of weirs on pervious foundations.

Water Cushion to Receive Falling Water.—The object of a water cushion is to protect the floor or stream bed against the impact of the falling water and to destroy the velocity as much as possible. The water cushion may be formed as a basin with its floor depressed below the surface of the stream bed by an amount equal to the desired depth of the water cushion, or may be formed above the stream bed by making the floor at the same level as the stream bed and using a secondary weir wall at the downstream end of the cushion, of sufficient height to give the desired depth of water above the stream bed. The first method requires excavation and floor construction below the stream bed where the ground water may increase the difficulties of construction and the cost. The second method produces a minor fall at the secondary weir wall, which may require greater protection of the stream bed below.

The length and depth of the water cushion may be determined from considerations similar to those given in the discussion of falls or drops on canals. The thickness of the floor of the water cushion, where no uplift pressure is effective, if of concrete, is commonly made equal to about $\frac{1}{10}$ of the height, but not less than 1 foot.

Water cushions are not often used for diversion weirs; the use of a strong floor without water cushion is usually depended on.

Riprap, Paving or Lining to Protect Stream Bed.—On loose foundations of sand, silt, or gravel the water in passing from the smooth surface of the floor to the surface of the stream bed has a tendency to wash away the material at the end of the floor, which may undermine this end of the floor and endanger the structure. To prevent this, the floor is continued with riprap, paving or lining for considerable distance downstream. The length and thickness of this riprap is considered in the design of weirs built on pervious foundation.

Impact of Ice, Trees, Etc., on Upstream Face of Weir.—To facilitate the passing of ice, trees and other floating débris, the crest of the weir on the upstream side is curved or built on a slope of 2 to 4 horizontal to 1 vertical.

DESCRIPTION OF DIVERSION WEIRS

Sand, Gravel, Cobblestone, Loose Rock and Brush Weirs (Plate II, Figs. A, B, C).—In this class are included the crude types extensively used in the early stages of development of many of the older systems and still used to some extent where irrigation practice is crude or where the capital is limited.

The body of the weir, which is seldom over 10 feet in height, is built with the material taken from the stream channel which is usually sand, gravel, or cobblestones. The most common form is a combination of this material with sage brush, willow or cottonwood branches. The brush serves to protect the material from being washed away by the current and the heavy material is necessary to obstruct the flow and hold the brush down.

The method of construction is well illustrated by the method used by the Yolo Water & Power Co. for the diversion of water from Cache Creek, California, until 1912 when a permanent concrete dam was built. The diversion dam, renewed each year after the danger of winter floods was passed, was built of mats of willow branches, weighted down with loose gravel and sacks filled with sand and gravel. The willow branches were first tied into bundles with galvanized iron wire, and each mat, about 16 feet square and 3 to 4 feet thick, was then formed by assembling these bundles, tying and weaving them together with wire. The mats were floated down by the current, held in position by ropes and weighted down with layers of gravel and sand about 1 foot thick. In this manner two to three tiers of mats were placed to bring the dam to the required height.

A different form of construction consists in driving posts in the stream bed, to which the brush is tied with wires or cables. The dam may consist of two parallel brush walls, with a filling of rock gravel or sand in between; the brush walls being formed by driving two parallel lines of posts and by tying or interlacing brush to each line.

A more crude form of dam, commonly used, is formed of layers of loose brush or branches, placed with the butt end downstream

FIG. A.—A brush dam on the Yellowstone River. (Bull. 249 Office of Expt. Sta., 1912, U. S. Dept. Agr.)

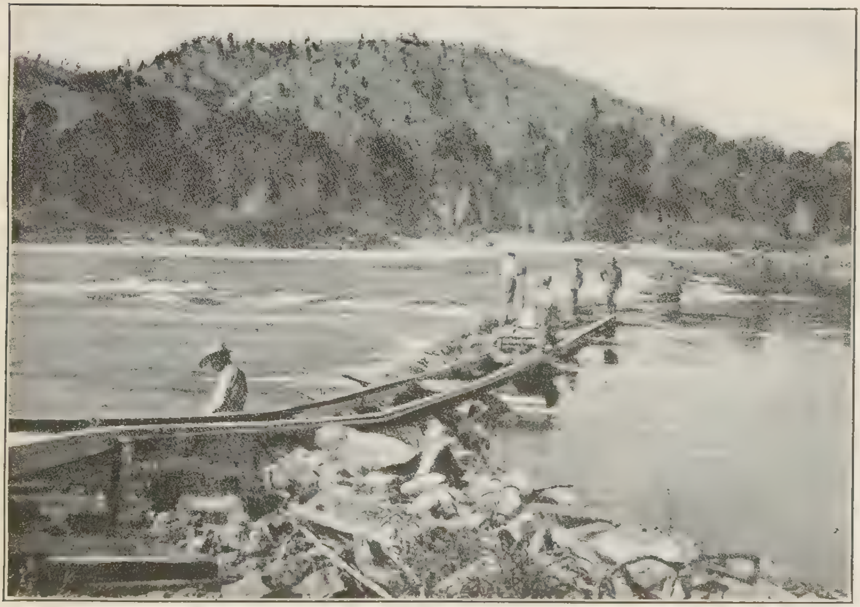


FIG. B.—Irrigation investigations. Repairing brush and rock dam. Yellowstone River, Mont. (Annual Report Office of Expt. Sta., June 30, 1902, U. S. Dept. Agr.)

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Plate II.

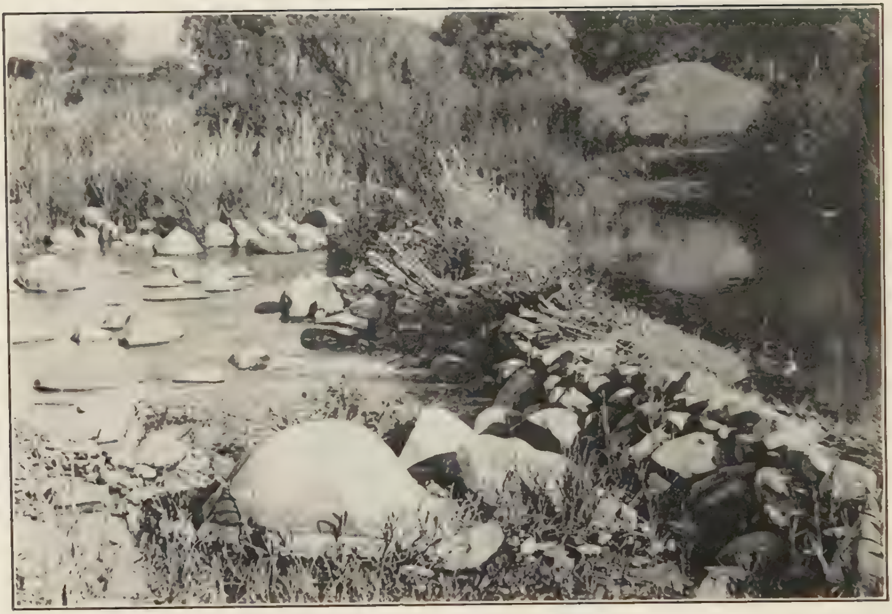

Fig. C.—Cobble and brush dam on Cache Creek near Rumsey, Calif.

Fig. D.—Small continuous crib diversion weir and fish ladder. Kamloops Fruitlands Irrigation & Power Co., B. C.

and weighted down with cobbles or gravel; this is illustrated by the cobble and brush weir formerly used at another point of diversion on Cache Creek, California (Plate II, Fig. C), and one on the Yellowstone River, Montana (Plate II, Figs. A and B). The placing of the butt end downstream favors the deposition of the silt and sand on the upstream side, in the brush and between the cobbles; which is necessary to make the dam more impervious.

This class of weir is not permanent and where the winter floods are heavy, even with the best construction, annual repairs or total replacement are necessary. It not infrequently occurs that unexpected late floods will wash out a dam recently replaced or repaired. A well-constructed weir of brush and cobbles or on a cobble or coarse gravel foundation may withstand the scouring effect of a moderate depth of water over it with little damage, but a weir built of smaller material and without brush, if submerged, will probably be entirely destroyed.

When the body of the weir is built of cobbles or rock and on a foundation of smaller material, the underflow through the pervious stream bed will have a tendency to wash out the finer material, as explained in the theory of the design of diversion weirs of the loose rock-fill Indian type, and there will be a gradual sinking of the cobbles or rock in the stream bed. The continued addition of new material to build up the weir will improve its strength, but it will not become stable and safe until sufficient material has been added to give it proportions comparable with those of the loose rock-fill Indian type and only provided the rock, boulders, or cobbles are sufficiently large to resist the transporting and erosive power of the flood water.

The yearly cost of renewal and maintenance of this type of diversion weir will usually make it more economical to construct a more substantial and permanent structure.

Log Weirs (Fig. 13).—This type of weir is suitable where timber is cheap and for heights of dam not greater than 10 to 15 feet. Its use is largely limited to certain projects in the Northwest, where the point of diversion is in the upper part of the stream, with timbered watershed, and where the difficulties of transportation preclude the economic construction of any other type of weir. The weir is built of layers of logs, placed side by side, with butt ends downstream. The weir cross section is triangular; the upstream face slopes at the rate of about 1 foot

vertically for 3 feet horizontally from the stream bed up to the crest, and the downstream face formed by the butt ends is nearly vertical. The bottom layers of logs extend downstream to form an apron for the protection of the stream bed against the scouring which would result from the impact of the falling water and from the irregular currents. The length of this apron measured from the toe of the downstream face should be from 3 to 6 times the height of fall, depending on the character of the stream bed.

Logs 10 to 20 inches in diameter are used, the larger logs are placed in the bottom. The downstream layer of the apron is first laid on the stream bed; the next layers of the apron overlap in turn, the upstream portion of the layer underneath, forming steps 10 to 15 feet wide. The triangular portion of the dam is then built up in layers, filling the spaces between with branches, brush, stone, and earth. To secure greater stiffness, cross binders

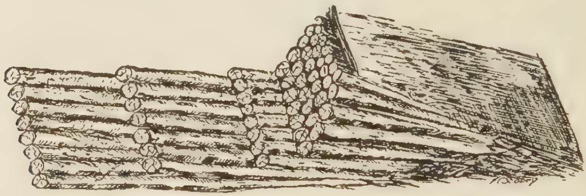

FIG. 13.—Sketch showing manner of placing logs in a typical log dam. (*Bull.* 244, Office of Exper. Sta., U. S. Dept. of Agri.)

3 to 4 inches in diameter are placed near the butt end of the logs, to which they are connected with tree nails or spikes. After the top layer is placed, several cross binders are secured to the logs and the surface is brought to a uniform slope with a filling of earth and rock.

Crib Weirs (Plate II, Fig. D and Plate III, Figs. A and B).—Dams built of log cribs filled with rock were extensively used in the early development of placer mines in California. These dams, usually built in the high mountains, were used for the storage of water required for hydraulic mining, and where this method of mining had to be abandoned on account of State laws the water has been conveyed to valley lands for irrigation. This type of dam or weir is also used quite extensively for the purpose of storage and diversion in the Northwest. They can be constructed in running water more easily than concrete or masonry dams, and for this reason have been used in a number of cases for temporary structures in the diversion of water to

FIG. A.—Storage overflow crib dam. White Valley Irrigation System, B. C.

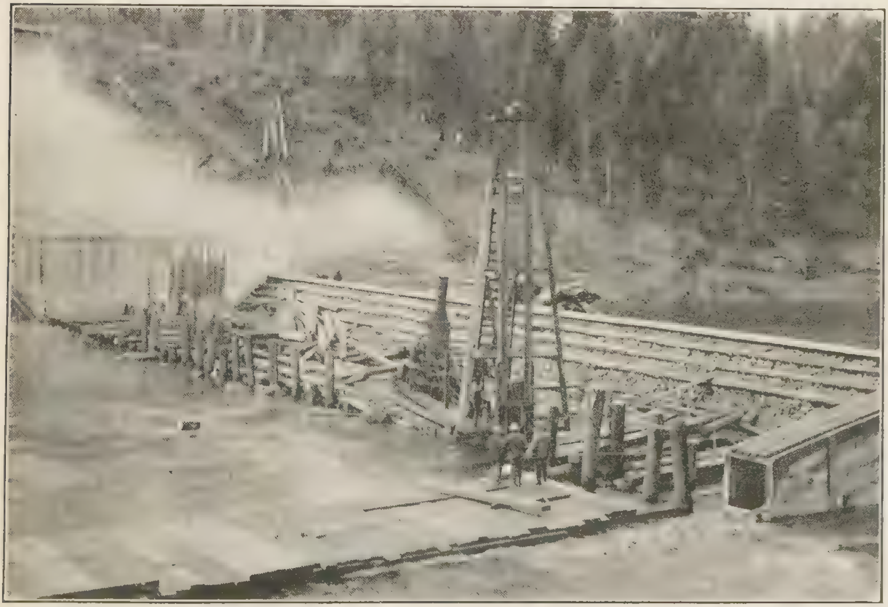

FIG. B.—Continuous crib spillway dam on Clealum River. Yakima Project, Wash. Showing rollerway decking removed to make repairs after failure of apron.

(*Facing page 44*)

Fig. C.—Wooden frame open weir on Fresno River near Madera, Calif.

Fig. D.—Moore collapsible diversion weir on Cache Creek, Calif.

permit the construction of more important concrete dams or weirs.

The weir consists either of crib work built continuously, or of separate cribs bolted or tied together with cables in rows or series. The compartments or pens formed by the logs are usually filled with rocks, cobbles, or gravel. This material gives to the dam the necessary stability against floating, overturning and sliding, and the crib work confines and holds together the heavy material, protecting it from the erosive and transporting power of the current. The upstream face of the weir is often vertical. A sloping face will increase the stability because of the downward component of the water pressure on that face, and a slope may be given which will give a downward pressure sufficient to make the weir stable without the filling of heavy material; this has been done in a few cases where the depth of water in the channel is small as compared with the height of the weir, so that the buoyancy of the submerged timber did not decrease the stability below a safe value. In which cases the weir has usually a triangular profile, with a sloping upstream face and a vertical downstream face.

The upper part of a rock crib weir may be sloped up to the crest to facilitate the passage of floating ice.

The design of the form of the weir will also depend largely on the character of the foundation and on the volume of stream flow. The foundation or stream bed must usually be protected from the erosion and impact of the falling water. For this the downstream face of the dam may be sloped or may be vertical with a direct fall on the foundation or protecting apron, or it may be stepped to break the fall. A sloping rollerway face is more difficult to construct and will not destroy the velocity as well as a direct fall or a stepped fall, and the erosive force of the higher velocity is a disadvantage where the stream bed consists of loose or soft material; on the other hand where the stream carries much ice, the impact of the ice is less destructive with the rollerway face.

The most common types of crib weirs are:

First.—The rollerway or sloping downstream face type which has a triangular profile, the upstream face being vertical or nearly vertical. This type is illustrated by that used for a temporary storage dam on Keechelus Lake, Washington (Fig. 14).

Second.—The stepped downstream face type formed by the

series of stepped compartments, illustrated by the temporary diversion dam on the Feather River in California (Fig. 15).

Third.—Sloping upstream face with a direct fall or steep downstream face as used for the diversion dam of the Bear River Canal, Utah (Fig. 16).

In the rollerway type a comparatively flat slope is desirable. Where the stream bed below is not solid rock, a slope not steeper than 4 feet horizontally to 1 foot vertically is preferable.

In the stepped downstream face the steps must be sufficiently wide to receive the sheet of falling water well inside of the outer edge; a width equal to at least 3 times the height of the step and preferably more for the lower steps is commonly used in practice. The width should also be dependent on the volume of water, and a consideration of the principles of flow over weir crests presented in preceding pages indicate that the width must be greater than obtained from the proportions stated above, when the depth of water over the crest is greater than about 4 times the height of fall or step. Safe dimensions would be obtained by using the following proportions: Use a minimum width of step of 10 feet; for depths of water over the crest not greater than 4 times the height of fall use a proportion of width to height of step of 3; and for depths of water over the crest between 4 to 6 times the height of fall use a proportion of 4. For the lower step use a proportion of 6. These dimensions will insure that the water strikes the steps well within the outer edge of the step.

Crib weirs to be built economically require cheap timber and rock or gravel; this is usually obtained in the upper part of the watershed of a stream where the stream bed is rock or cobbles. Where the bed is hard compact rock, it may not be necessary to use a protective apron, but where the bed is softer material an apron must be provided; this is formed by extending the lower portion of the cribs a safe distance beyond the erosive effect. On a rock foundation and where the running water may be controlled so as to permit it, the lower timbers of the cribs may be secured to the rock with 1-inch anchor bolts spaced 6 to 8 feet apart, set in cement mortar, or trenches may be dug in the rock bed to receive the lower timbers. To obtain a water-tight connection with the impervious rock, the junction along the upstream toe is usually made by extending the timber toe in a shallow trench, which is then filled with concrete.

On loose foundations of cobbles or gravels it is desirable to excavate the area of the river bed, on which the weir is to be built, to a depth of at least 4 to 6 feet below the natural surface, and extend this excavation to the lower edge of the apron, in order to permit the construction of a rock-filled crib apron 4 to 6 feet thick. To give the weir additional anchorage in the foundation, the first row of compartments formed by the cribwork at the upstream toe of the weir and also the last row of compartments at the downstream end of the apron may extend at least 6 to 8 feet deeper in the stream bed. Where an impervious stratum may be reached at a moderate depth, 20 feet or less, the underflow is intercepted by driving along the upstream toe steel or wooden sheet piling. Where an impervious stratum cannot be reached, the principles of underflow, as presented in preceding pages on the principles of design of weirs on pervious foundations, should be considered. In these principles are also presented the requirements of a cut-off wall at the downstream toe of the apron to prevent undermining by backwash and of a riprap or paving protection for the stream bed extending beyond the end of the apron.

The cribwork is made either of round, hewn or sawed timbers. When made of round or hewn timbers, logs 10 to 20 inches in diameter are used. When made of sawed timbers, 10 to 12-inch timber is used. The compartments are usually square and formed by spacing the timbers from 6 to 12 feet apart.

In continuous cribs the timbers running along the axis of the dam are the stringers. Each stringer is carried continuously across the stream and is built of timbers joined at the ends. Either a butt joint with a bolted splice on one side or a bolted lap-spliced joint may be used. The cross pieces or ties and the stringers are placed alternately in courses. When logs are used, notches are made at the intersections; these are not necessary for sawn timber. The timbers are secured together by $\frac{5}{8}$ to $\frac{7}{8}$-inch drift bolts. The compartments formed by the cribwork are often filled with rock or cobbles of such size that can be handled by one man, although larger quarried rock put in place by a derrick may be used; gravel and small cobbles are also desirable material; finer gravel or sand may be used, but is more liable to be disturbed or washed out through the cracks in the decking.

The cribs when filled are covered with a decking. The up-

stream face must be water-tight; for this, a double layer of 2 or 3×12-inch lumber is commonly used, and in some cases a layer of roofing material in between has been used. To increase the impermeability, an earth facing is placed against the upstream face, sloping on a 2 to 1 slope to the crest of the dam. The decking for the downstream face or for the surface of the steps is made of one thickness of heavier lumber; 4 to 8 inches thickness is commonly used.

Crib weirs built of separate compartments are not as commonly used as continuous cribs. They are used to advantage when it is necessary to build the weir in running water. The separate cribs or compartments are made with bottom timbers laid close together to hold the rock or material used to sink them. The cribs are floated and placed in position, they are then filled only sufficiently to make them sink; this obstructs the flow less than if they were entirely filled and facilitates construction in the running water. The separate cribs are placed in rows and steps are formed by two or more rows adjacent to each other and the cribs are then secured together with cables. When constructed in running water, the cribs in sinking increase the current under them; this washes out some of the surface of the stream bed down to a depth where a comparatively firm foundation is obtained. When the cribs are to rest on a rock foundation, the bottom of the cribs may be fitted to the surface of the foundation by taking soundings and building the cribs accordingly. When the cribs are all in place the filling and decking is completed.

Crib Spillway Dam on Keechelus Lake, Yakima Project, Washington (Fig. 14).—This dam was built for a temporary storage overflow dam on Keechelus Lake at the headwaters of the Yakima River in the Cascade Mountains, Washington. The length of the dam with the earth embankment extensions on both sides is about 256 feet, and between the crib abutments is about 190 feet. The foundation is gravelly with some boulders, and at the site was cleared and stripped of loose material to a depth of about 2 feet. The maximum height is about 14 feet. The cribwork consists of logs not less than 10 inches in diameter at the small end, notched at the intersections to a vertical thickness of 8 inches, secured to each other with 1-inch drift bolts 16 inches long, driven in holes $\frac{1}{8}$ inch smaller. The compartments are about 8 feet square and were filled with gravel and

rock, except at the aprons where the filling was made entirely with rock, of which 75 per cent. had to be at least $\frac{1}{10}$ of a cubic foot. The sheathing on the upstream vertical face was made of two thicknesses of $1\frac{1}{2} \times$ 12-inch plank, laid vertically with lap joints and extending down into a cut-off trench 4 feet deep, refilled with selected material. The rollerway and apron were covered with 6-inch hewed timbers with $\frac{1}{2}$-inch open joints, and secured to the logs with $\frac{5}{8} \times$ 13-inch drift bolts. A clay and gravel embankment built against the upstream sheathing, with a slope of 2 to 1, increases the path of percolation under the

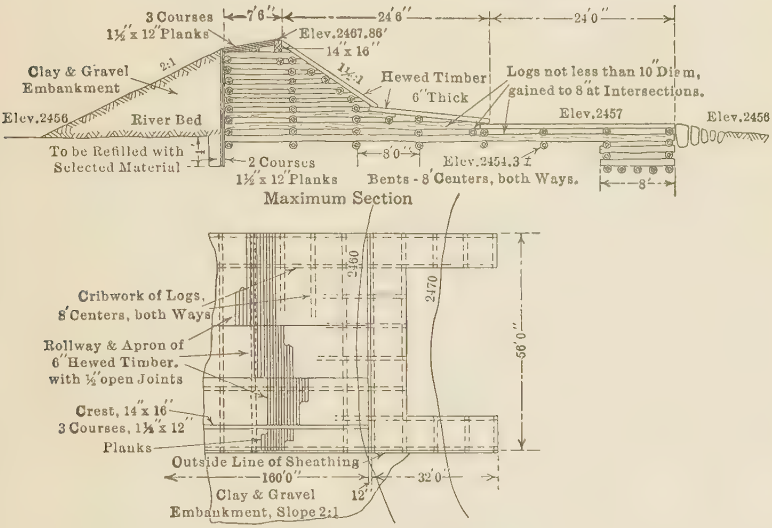

FIG. 14.—Crib Dam. Lake Keechelus. Yakima Project, Wash.

structure, and is desirable for a weir built, as in this case, on pervious material.

On another dam of practically the same dimensions, the Clealum dam (Plate III, Fig. B), the flooring and some of the upper logs of the apron separated and floated up, indicating that this part of the dam had not sufficient strength or weight. This may have been due to defects in construction, such as making the holes for the drift bolts larger than specified, in order to facilitate the driving of the bolts. To obtain greater weight, a lower set of logs should be placed sufficiently close together to hold the rock on them and the decking and logs carefully se-

cured to make the entire cribwork hold together. A suggested alternative would consist in forming the downstream cut-off anchor cribs at the foot of the rollerway instead of at the downstream end of the apron and making the apron a loose floating one.

Temporary Crib Diversion Weir on Feather River, Calif. (Fig. 15).—This weir was built by the Great Western Power Co., on the North Fork of the Feather River, preceding the construction of a larger permanent diversion dam. The crib weir has an upstream slope of 2 to 1, a level crest 10½ feet wide, and a stepped downstream face of three steps, each 16 feet wide. The crest of

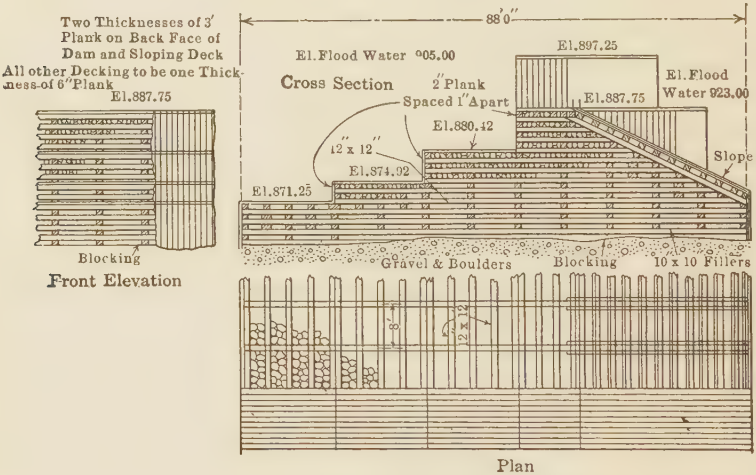

Fig. 15.—Details of temporary timber dam on Feather River, Calif. (*Eng. Contr.*, Oct. 16, 1912.)

the weir is 16½ feet above the floor of the lower step, and provision was made to raise the crest with 8 feet of flashboards. The weir is 280 feet long, built on solid rock at each end, and on boulders and hardpan in the center. Timber sheet piling was driven along the toe of the upstream face. The body of the weir is built of 12 × 12-inch square timbers, bolted together with ⅞-inch drift bolts, 26 inches long. The deck of the upstream face is made of two layers of 3-inch lumber; the crest and the floor of the steps are planked with one layer of 6-inch lumber.

During the first winter the weir withstood a flood flow of 95,000 second-feet, producing a depth of water on the crest of about 22 feet. This flow was about ¾ of the maximum expected

flood flow. The only damage was the wearing of the deck in places from 2 to 4 inches, due to the sand-blast action of the large amount of silt in the water.

Crib Weir on Bear River, Utah (Fig. 16).—This weir diverts the water of Bear River into the two main canals of the Bear River Canal system (Utah Idaho Sugar Co.). It is a continuous crib, 370 feet in length, with a maximum height of 17½ feet. The weir is built on and anchored to a rock foundation for about

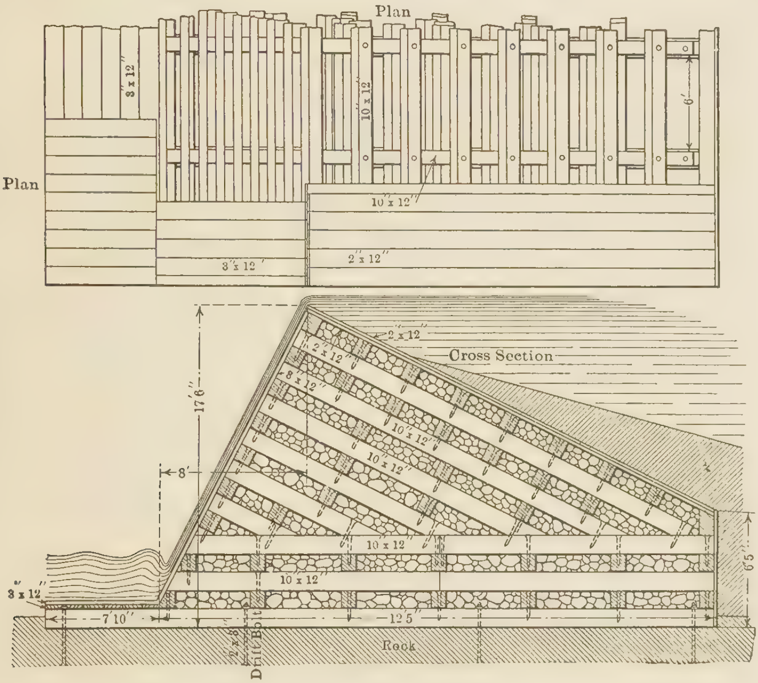

FIG. 16.—Crib weir of Bear River Canal, Utah.

⅔ of its length; the other ⅓ is built on clay. A leak through the clay gradually undermined the weir, allowing the entire flow of 20,000 second-feet to pass under the weir. This left a length of 60 feet of the weir spanning over the cut for a period of 6 months, without any damage to the body of the weir. The cut was closed with a concrete wall 4 feet thick, about 15 feet high, connecting the toe of the crib, along the upstream face, with the rock foundation, and the balance of the cut was filled with rock. The weir was built in 1890 at a total cost of about

$45,000, the timber alone costing $22,000. Some of the timbers have rotted and had to be replaced as early as 1902. In 1907 it was deemed desirable to relieve the pressure on the weir during flood flows by removing the upper 5 feet of the crest and replacing this portion with collapsible gates of the type described farther.

Wooden Frame Open Weirs (Plate III, Fig. C).—This type of weir has been extensively used in the San Joaquin Valley, California, for the diversion of water from some of the most important rivers. They have usually been built on the lower portion of the stream, where the stream bed is sand or small gravel, and where an open type of weir was necessary to offer little obstruction to the flood flow and prevent the overflow of lands above. These locations require a comparatively low weir, the heights which have been used being usually under 10 feet.

The weir is formed essentially of a wooden floor anchored usually by sheet piles and anchor piles to the stream bed and of a framework built on this floor, consisting of bents, spaced usually from 6 to 8 feet apart, framed together by longitudinal pieces, including also the operating platform. The waterway is thus divided into panels or bays, and the flow through them is regulated by horizontal flashboards, placed in the grooves formed on the upstream edge of each bent. The flashboards are inserted in or removed from the grooves from the operating platform. On account of the superstructure, this type of weir is not suitable on streams which carry large floating material.

The low first cost is the chief advantage and is important in considering more expensive types of weir. The larger part of the cost is in the substructure, and this part is practically permanent, as it is usually kept constantly wet by the underflow and the water dammed up. The superstructure will need occasional renewals, or may be damaged by unusual floating material, but the cost of renewals or repairs will be comparatively small.

The design of these weirs has been based largely on practical results and varies considerably with the experience and judgment of the engineers who have planned them. In some cases it is apparent that just criticisms may be made, and that by applying the principles presented by Mr. Bligh on the design of weirs on pervious foundations many improvements could be made.

The stability of the superstructure against overturning depends largely on the downward component of the water pressure on the

sloping face formed by the flashboards; the slope given to this face varies from about 1 vertical on 1 horizontal to 2 vertical on 1 horizontal. The flatter slope gives greater stability, but is not so favorable to the operation of flashboards. The most important part of the design is the connection and anchorage of the substructure with the sandy stream bed. It is necessary to make the path of percolation around cut-off walls and under the floor of sufficient length, and the floor must be anchored to resist the upward pressure and flotation.

Weir on Kern River, Calif. (Fig. 17).—This weir used on the Kern River for the diversion of the Beardsley canal is 204 feet

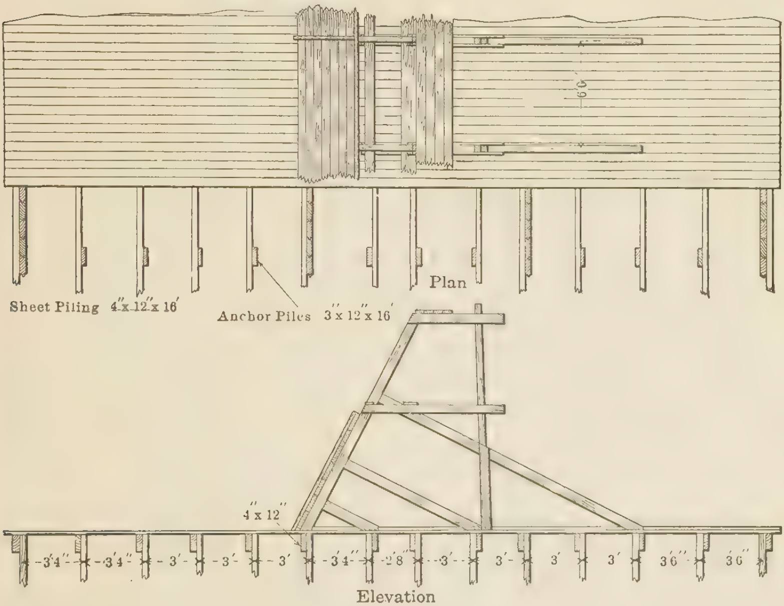

FIG. 17.—Kern river weir. Beardsley Canal, Kern Co., Calif.

long between abutments. The design shows four sheet piling cut-off walls which with the floor give a path of percolation of about 180 feet; this is apparently larger than is required by Bligh's theory. The second and third row of sheet piling could have been omitted without making the structure unsafe. The large path of percolation is, however, advantageous in decreasing the amount of underflow, which is desirable where the supply is limited. The upward pressure on the floor is resisted by the anchorage provided by the sheet piling and anchor piles. The

2-inch floor is nailed to the stringers, parallel with the axis of the weir, spaced about 3 feet apart, and the stringers are secured to the top of the anchor piles and sheet piling. The anchor piles in the upstream four rows are spaced 12 feet apart in the row; in the succeeding downstream rows the anchor piles are alternately 6 feet and 12 feet apart. The upward pressure on the portion of the floor upstream from the upstream toe of the bents is more than balanced by the downward pressure of the water.

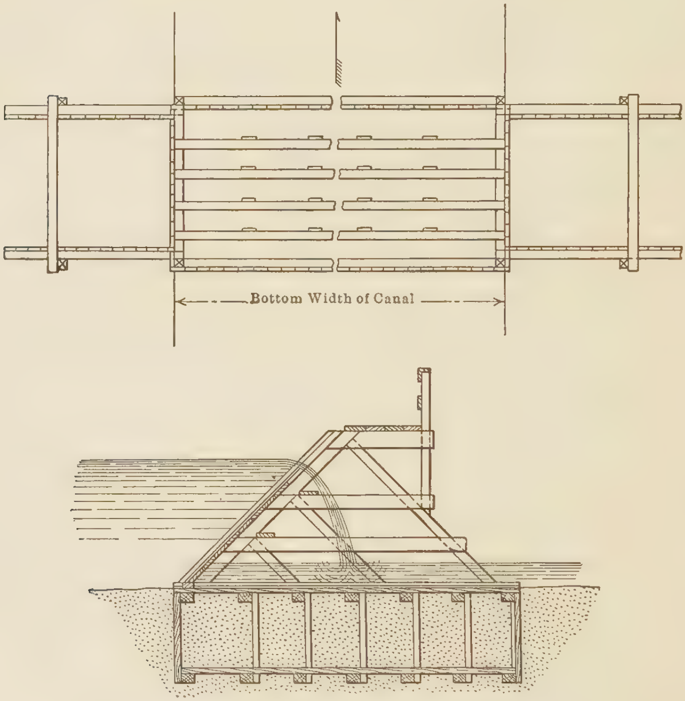

FIG. 18.—Type of wooden frame diversion weir by W. C. Hammatt. (*Trans.* Am. Soc. C. E., Dec., 1913.)

The only purpose which this portion of the floor fulfills is to increase the length of the path of percolation. The smaller number of anchor piles used for this portion of the floor is, therefore, desirable, but it is questionable whether any of these upstream anchor piles are needed, unless it be to hold the floor level in case there is a shifting of the fine sand under the floor or to facilitate the placing of the stringers and floor.

W. C. Hammatt has presented in an article in the Transactions

of the Am. Soc. of C. E. of Dec., 1913, on *A Western Type of Movable Weir Dam* several forms of this type of weir. In one form (Fig. 18) the anchorage is obtained by connecting the floor to a lower floor built 4 to 6 feet below it, with upstream and downstream walls forming an anchor box, which is filled with sand. Mr. Hammatt, after a consideration of the different designs, submits an improved design (Fig. 19) in which the upstream anchor piles are omitted and the downstream portion of the floor is made with open joints to relieve the upward pressure on the floor. In fine sandy beds the open joints may not be desirable, as it may permit the blowing through of the sand.

The object of the lower downstream sheet piling cut-off wall

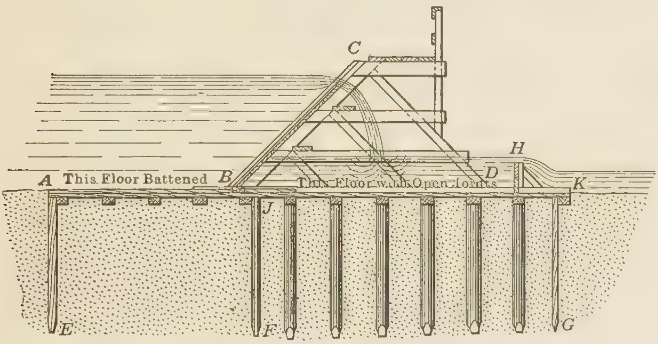

Fig. 19.—Improved type of wooden frame diversion weir by W. C. Hammatt. (*Trans.*, Am. Soc. C. E., Dec., 1913.)

is mainly to prevent undermining; for this purpose it could be made of less depth than the upstream cut-off wall.

The same type of weir can be built of reinforced concrete, and where an open weir is not necessary a solid face could be used in the place of the removable flashboards.

Concrete and Masonry Weirs.—Solid masonry or concrete weirs of the gravity type have the advantage of strength and durability, but are usually more expensive than the other types of weir, although this will depend on the availability of the materials. The best foundation is solid rock, but they are not limited to this type of foundation. They have been constructed on loose foundations in India and a number of them have been constructed on foundations other than solid rock by the United States Reclamation Service.

The weir consists of a weir wall and, in the case of soft founda-

tions, requires also a floor, one or more cut-off walls and a downstream protective apron. The principles of design of the different parts have been fully discussed in preceding pages. Two general forms may be considered, depending on the shape of the downstream face: the Ogee form and the direct fall on the solid foundation or on a strong floor. The first form is illustrated by the diversion weirs for the Yakima project in Washington, the North Platte project in Wyoming-Nebraska, the Boise Payette project in Idaho, the Rio Grande project in Mexico, the Granite Reef in Arizona. The second form is illustrated by the diversion weir of the Yolo Water & Power Co., on Cache Creek, California, and the Burra weir in India.

Weir on Yakima River, Washington (Fig. 12*A*).—The Yakima project diversion weir is described in the general description of the headworks of this project.

Weir on North Platte River, Nebraska (Fig. 12*B*).—The North Platte project diversion weir is built on a foundation of rock conglomerate to a depth of at least 6 feet, underlaid by clay. The height from the rock surface to the crest varies from about 15 feet near one abutment to a maximum height of about 32 feet near the other abutment. The cross section is of the Ogee type. A concrete apron of a minimum thickness of 12 inches extends for about 80 feet downstream from the toe of the Ogee. The connection with the rock foundation is made with trenches excavated in the rock, and to relieve the uplift pressure on the base of the dam a 6-inch tile drain is placed in a trench, excavated parallel to and near the upstream toe of the dam. The tile is surrounded with gravel and discharges through 6-inch tiles placed 25 to 30 feet apart. The diversion weir connects at each end with the headgates of the canals on each side of the river. On one bank of the river the headgates and canal are in cut; on the other the bank is low and requires an earth embankment to connect the headgates with the sloping river bank.

Weir on Boise River, Idaho (Fig. 12*C*).—The diversion weir of the Boise Payette project is built on hard compact gravel. The length of the weir crest is 216 feet, and at one end is provided a logway 30 feet wide. The estimated maximum flood flow was 40,000 second feet. The height of the weir crest above the floor is 35 feet. The Ogee weir wall is built of concrete with imbedded rock. The face is uncoursed rubble masonry and the curved crest is faced with concrete at least 1 foot thick. The crib timber

apron is made of timber sawed square and filled with stone and gravel.

Weir on Rio Grande River, New Mexico (Fig. 12*D*).—The diversion weir of the Rio Grande project, known as the Leasburg weir, is built on a pervious foundation. The weir is 600 feet between abutments; it consists of the weir wall and a reinforced concrete floor, resting on piles, a wooden sheet piling cut-off wall 20 feet deep at the upstream toe to increase the path of percolation against underflow, and a wooden cut-off wall 5 feet deep at the downstream end of the floor to protect against undermining. Weep holes just upstream from this second cut-off wall decrease the upward hydrostatic pressure on the floor. The upper face of the floor is 2 feet below the mean river bed; this insures a water cushion of at least that depth and increases the stability of the floor against upward pressure. The pile foundation consists of parallel rows of piles across the river, 4 feet apart, except for the first three rows (3 feet 5 inches apart). The piles in each row are 4 feet apart and are staggered with those of alternate rows. The purpose of these piles is apparently to give anchorage against sliding on the base, and to support the floor against downward pressure and impact, to resist which the floor is reinforced. If the top of the piles were secured to the reinforcement, additional strength would be gained against upward pressure, but as the weir crest is only 5 feet above mean river bed, this added strength is not needed. One end of the weir joins to a rock abutment, while the other end is formed with a concrete abutment which joins to an earth embankment extending for about 1,600 feet up to the necessary height on the bank of the river.

Weir on Salt River, Arizona.—The Granite Reef diversion weir is on the Salt River, Arizona, about 20 miles east of Phoenix. It was built in 1908 by the U. S. Reclamation Service for the diversion of water from the Salt River to irrigate the lands included in the Salt River project. The natural flow of Salt River at the point of diversion is subject to great variations, from a minimum of 55 second-feet to a maximum which may have been as much as 200,000 second-feet before the development of large storage above, by the construction of the Roosevelt dam. The diversion weir was designed for a maximum expected flood flow of 165,000 second-feet. The river carries at times large quantities of silt and sand. The weir is 1,000 feet long and con-

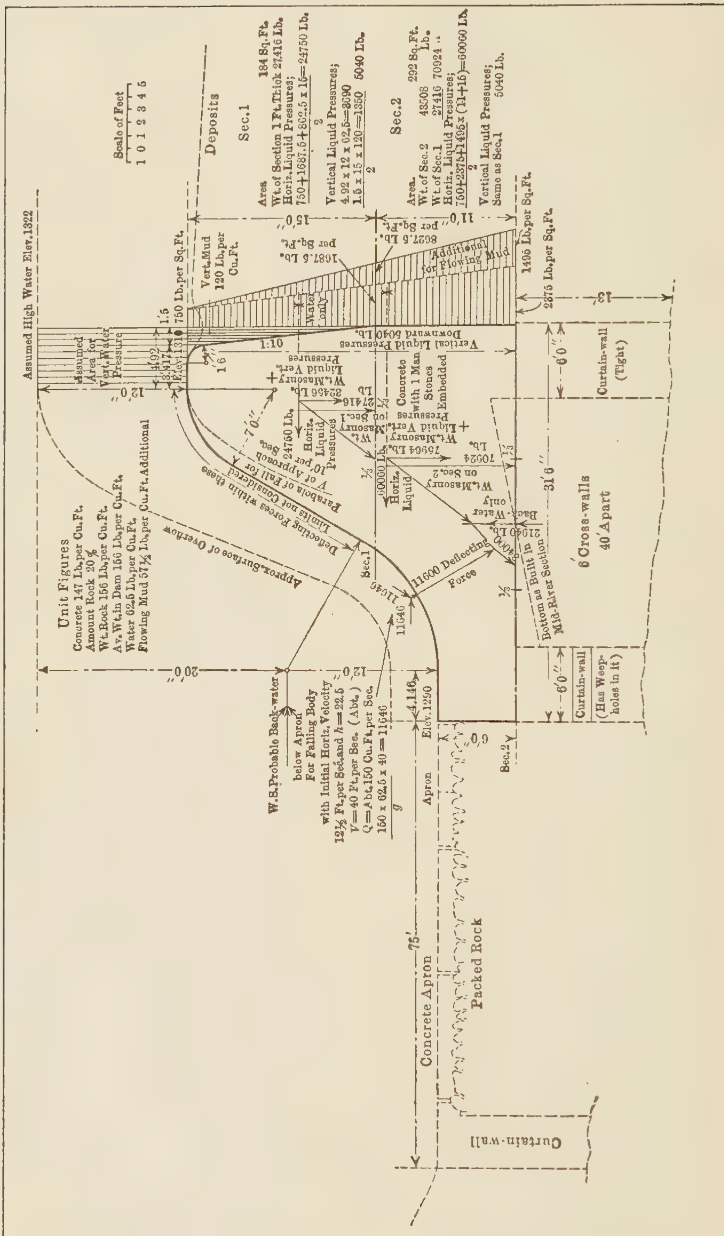

FIG. 20.—Analysis of Granite Reef dam, Salt River Project, Ariz. (*Trans.*, Am. Soc. C. E., Dec., 1914.)

nects at each end with a sluiceway built directly in front of the canal headgates; there being one canal on each bank. The stream bed on the site of the diversion weir is sand, gravel and boulders underlaid with granite rock at varying depths. For about $2/3$ of the length of the weir the rock was at moderate depth, which made it feasible to support the body of the weir on the rock foundation by means of supporting walls and piers extending from the rock to the base of the weir (Fig. 20). These foundation walls consist of two parallel longitudinal walls across the river, each 18 inches thick, one under the upstream toe of the weir wall and the other under the foot of the Ogee curve, spaced 29 feet center to center, and of cross piers 3 feet 6 inches thick, 20 feet center to center. The rectangular compartments formed by these walls hold the natural stream bed material or loose rock fill. The base of the weir wall is carried 11 feet below the natural river bottom, which brings it on the coarser, firmer material. The upstream longitudinal wall is intended to be an impermeable cut-off wall, the other longitudinal wall is pierced with drain holes to relieve the hydrostatic uplift pressure on the base, caused by any water which may creep inside.

For about 300 feet toward the center of the stream bed the rock was at too great a depth to be used for foundation. This part of the weir was built on a similar system of foundation walls, made thicker and extending down to a bed of compact gravel and stone at a depth of about 20 feet below the original stream bed. The two longitudinal walls are spaced 26 feet center to center and the cross walls 40 feet apart; all these foundation walls are 6 feet thick. The downstream longitudinal wall is pierced to relieve the uplift pressure.

The body of the weir or weir wall is a gravity dam, modified to form the Ogee curve. The crest of the weir is 20 feet above the upper surface of the apron, which is depressed about 5 feet below the natural stream surface to form a water cushion or basin at the foot of the Ogee.

The analysis from which the design of the weir cross section was determined by Mr. A. L. Harris, of the Reclamation Service, is indicated on the accompanying graphical presentation (Fig. 20). The design is worked out by considering the cross section in two sections, divided by a horizontal plane 15 feet below the crest. The resultant of all the forces acting on each section is determined and must cut the base of each section

within the middle third. In addition to the hydrostatic pressures on the crest and upstream face of the weir wall usually considered in the design of diversion weirs, as previously explained, in this particular design, a hydrostatic flowing mud pressure on the upstream face of the weir wall has been considered. This mud pressure is taken as an added pressure, from the crest down, equivalent to that of a liquid weighing 57.5 pounds per cubic foot. The pressure of the downstream backwater is not considered, but the resultant of the deflecting force on the lower curve of the Ogee is considered. The addition of the flowing mud pressure in the consideration of the forces determining the design of the cross section increases the stability of the structure. Whether this is necessary or not is questionable. The material deposited will be more or less compacted, especially near the bottom where the porous stream bed gives drainage to the deposited material. This action may decrease the water pressure on the upstream face. Also, the silt and finer material carried by the underflow and deposited in the pores of the stream bed material will increase the resistance to percolation and decrease the uplift pressure.

The apron is an important part of the diversion weir; it is 75 feet wide from the toe of the Ogee curve to the downstream cut-off or curtain wall. This wall is 4 feet thick and extends 12 feet deep into the stream bed to protect the end of the apron from undermining. The apron extends for the entire length of the dam, except where the outcrop of bedrock comes to the surface; at these outcroppings the base of the weir wall is built directly on the rock and no apron is necessary. The apron protects the soft stream bed against erosion. It is made of concrete, 18 inches thick, constructed in blocks 10 feet square, with open joints 3 inches wide. In the concrete are embedded boulders about 5 feet thick, placed so as to leave projecting surfaces which form good anchorage against sliding. The open space between slabs relieves the water pressure and gives a more elastic apron than a solid continuous concrete apron. On account of these open joints the length of the floor cannot be considered as part of the path of percolation, which measures about 84 feet and gives a percolation coefficient or ratio, with

the head of water of 20 feet, of only 4.2. This coefficient is small but the resistance to percolation is increased by the deposit of silt and sand on the upstream side of the weir. As small a path of percolation as provided in this case would not be desirable with an open joint apron, if the material of the stream bed were entirely sand, for the velocity of percolation would then be sufficient to carry the sand through the open joints. After the weir was first constructed, water percolating under the dam boiled up through the open joints, but the seepage channels were nearly all sealed during the first season by the silt deposited on and carried in the porous stream bed material. The maximum flood flow obtained was 6 months after completion when a depth of 7.2 feet of water on the crest of the weir was reached.

Mr. A. L. Harris, who made an inspection of the diversion dam in August, 1913, states that after the 5 years since its construction there is very little evidence of wear, produced by the large volume of silt and sand carried by this river. The crest of the weir was constructed of 1 :2 :4 gravel concrete. The contraction cracks which have occurred are practically all located at the construction joints which formed a V-tongue and groove with projecting steel anchor rods; there are 18 contraction joints in the length of 1,000 feet, or a distance apart of about 60 feet. A small amount of seepage water, probably less than ¼ of a cubic foot per second, discharged through the floor joints. No damages have occurred to the structure except the undermining of a few slabs of the floor, adjoining the projecting ledge of rock, and the damage was probably due to the erosion of the rock itself, which is a coarse-grained friable granite.

Capay Weir on Cache Creek, Calif.—This diversion weir of the Yolo Water & Power Co. was built on Cache Creek, California, in 1912. It is 500 feet long between abutments. Each abutment is built with the headgates to the canals diverting water on both sides of the river (Plate IV, Fig. A). The main parts of the weir are the weir wall and the apron (Fig. 21). The stream bed is gravel underlaid with indurated clay or hardpan. On the site of the weir the gravel was excavated and the weir wall built on the indurated clay. At the upstream toe of the weir wall a reinforced concrete cut-off wall was carried into the clay to stop underflow and to give increased stability against sliding; at the downstream toe a second cut-off wall

was carried in the clay; this wall is pierced with weep holes to relieve any underpressure which might be caused by the water creeping past the upstream cut-off wall. The apron is of reinforced concrete, 15 inches thick, and terminates in a third cut-off wall pierced with weep holes.

The weir wall is trapezoidal, with a comparatively steep downstream face, which gives to the sheet of overpour water a direct fall, except for small depths of water on the crest of about 1½ feet or less.

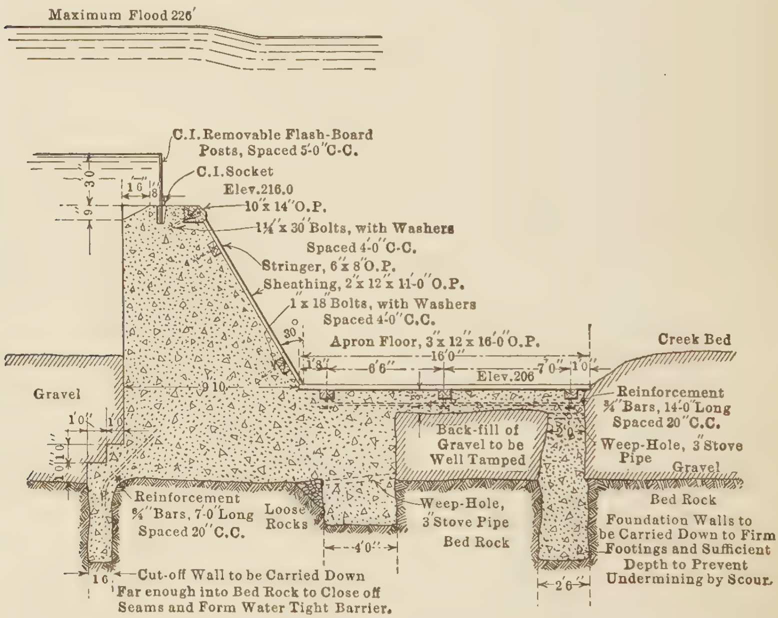

FIG. 21.—Cross section of Capay diversion weir. Yolo Water & Power Co. Calif.

The estimated maximum depth of water considered in the design was 10 feet. The low bank on one side of the river and the extent of flooded land on the upstream side of the weir during flood flows determined the maximum elevation of the weir crest; this was made 2½ feet lower than the full supply water level in the canals. To divert the full supply in the canals when the period of high stream flow had passed, cast-iron sockets, spaced 5 feet on centers, were placed in the crest of the dam to receive removable cast-iron posts forming grooves for the insertion of horizontal flashboards up to an added height of 3 feet. The down-

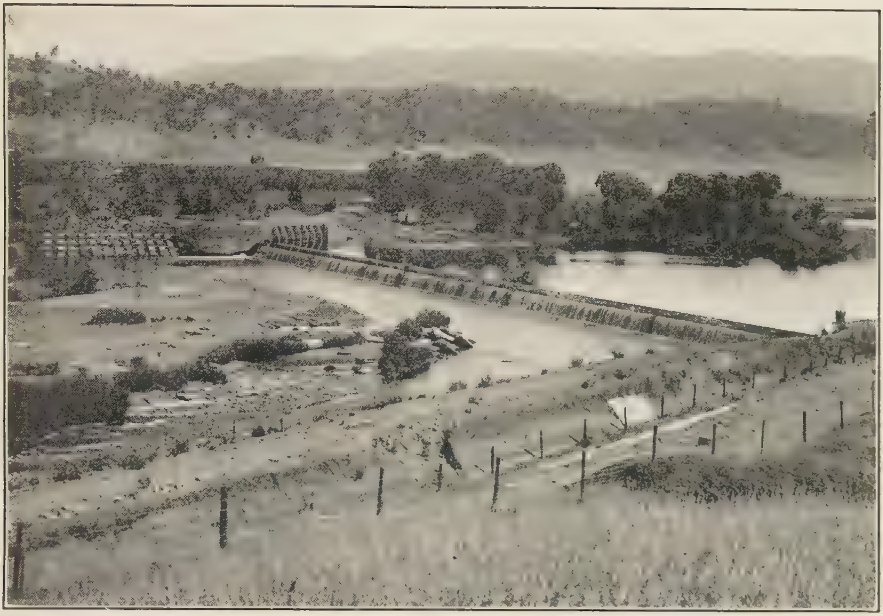

FIG. A.—Capay diversion works on Cache Creek, Calif. View shows brush revetment weighed down with concrete blocks. Yolo Water & Power Co., Calif.

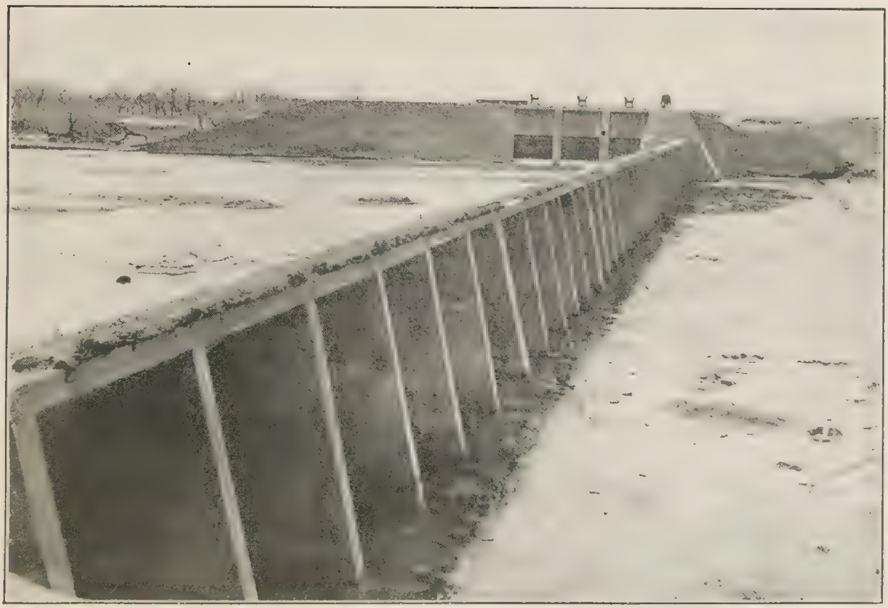

FIG. B.—Flat deck Ambursen type of reinforced concrete diversion weir. Nile Irrigation Project, near Ft. Morgan, Colo.

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PLATE IV.

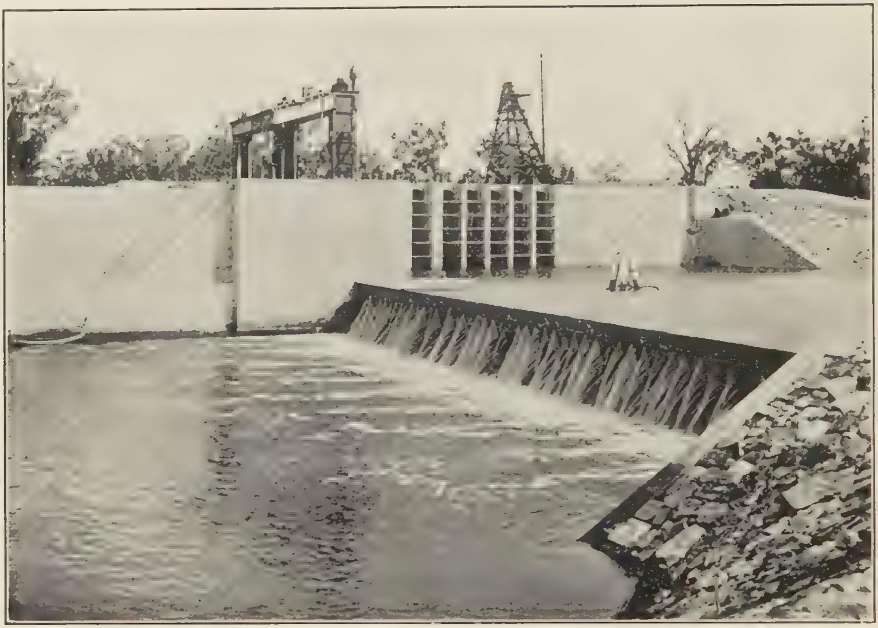


FIG. C.—Collapsible diversion weir on Murrumbidgee River, New South Wales, Australia.

FIG. D.—Same as Fig. C, with no stream flow and one shutter down.

stream edge of the crest of the dam is formed by a 10 × 14-inch piece of Oregon pine and the downstream face and apron are lined with 2 × 12-inch Oregon pine, intended as a protective lining against the erosion and wearing effect produced by the sand, gravel and cobbles carried during flood flows.

During the spring of 1914 a flood flow of considerable magnitude passed over the weir, producing a maximum depth of water on the crest of about 6½ feet, with no harmful effect on the weir. No special construction joints were made, and the contraction cracks which have developed are distributed at irregular intervals varying from 25 to 75 feet; these cracks are narrow and do not in any way weaken the structure.

Burra Weir, India (Fig. 22).—The Burra weir on the Mahanudde system in India represents a weir of the type with a direct fall on a floor or apron. The upper surface of the floor is raised above the downstream low water level. The advantage

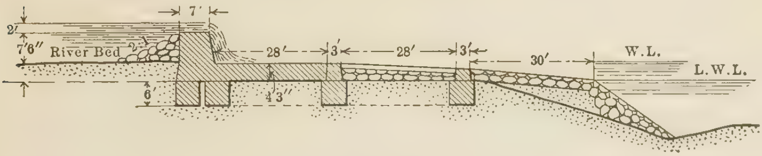


Fig. 22.—Burra weir, India.

of a raised floor of this type over that of a depressed floor is the decrease in uplift pressure and the decrease in the height of fall. The total head from the weir crest to the low water level is 12 feet; Mr. Bligh states that for the stream bed material under this weir the coefficient of percolation should be 12, and therefore the path of percolation should be 144, while it is actually only 112. Mr. Bligh suggests the following improvements in the design:

First.—Make the masonry apron 42 feet long from the downstream toe of the weir wall to the end, with a thickness of 4 feet at the weir wall tapering to 3 feet at the end.

Second.—Use sheet piling cut-off walls in the place of the foundation blocks, one cut-off wall to be directly under the center of the weir wall and to extend to a depth of 14½ feet below the base of the weir wall, the other cut-off wall to be at the lower end of the floor and to extend 12 feet below the underface of the floor.

Third.—To give the necessary length of path of percolation,

add an upstream clay puddle floor on the river bed, extending for 48 feet upstream from the weir wall.

Fourth.—Protect the stream bed for 90 feet downstream from the end of the masonry floor with a thickness of riprap or paving tapering from 4 feet to 3 feet.

Reinforced Concrete Weirs.—The use of reinforced concrete permits a variety of designs of diversion weirs, in which, as compared to the gravity type of masonry or concrete weirs, economy in the volume of concrete may be obtained. Greater elasticity is also an advantage of special value on foundations other than solid rock. On the other hand the cost of labor per cubic yard of concrete is greater and more careful construction is required. The selection will, therefore, depend on the availability of the material, the skill of the labor obtainable and the character of the foundation. The greater weight of gravity weirs and the comparatively recent use of reinforced concrete in general building construction probably accounts for the comparatively few reinforced concrete diversion weirs and dams.

The usual type of reinforced concrete weir is similar to the wooden-frame type of diversion weir; its stability depends largely on the downward component of the water pressure on a sloping face, supported on concrete buttresses. The buttresses rest on solid rock or on a strong floor, which, when built on loose material, must extend sufficiently far downstream to protect the stream bed. The sloping face may be a flat reinforced concrete slab, with the amount of reinforcement decreasing from the bottom of the slab to the crest according to the water pressure; or the face may be formed of a series of thin concrete arches supported on the buttresses. A special design of the concrete multiple arch type has recently been used for a storage dam having a maximum height above bedrock of 92 feet and a total length of 363 feet, divided by 11 buttresses in 10 spans of 32 feet each. This damwas built for the Bear Valley Mutual Water Co. in Southern California.

The Corbett diversion dam, constructed by the U. S. Reclamation Service on the Shoshone River, Wyoming (Fig. 23), and the Three Mile Falls diversion weir on the Umatilla River, Oregon (Fig. 24) built by the U. S. Reclamation Service, illustrate the flat deck and arched deck type respectively.

A flat deck Ambursen type of reinforced concrete diversion weir is illustrated by the Wiggin diversion weir of the Nile Irrigation

project, on the Bijou River near Ft. Morgan, Colorado (Plate IV, Fig. B). Its height is 9 feet and its length 532 feet; it is built on a sand and gravel foundation.

Corbett Diversion Dam, Shoshone River, Wyo.—The Corbett diversion dam, 400 feet in length, consists of a flat reinforced deck on a 1 to 1 slope, supported on piers, a reinforced floor, an upstream and a downstream cut-off wall at the upstream and downstream edge of the floor, and a foundation wall near the center line of the floor. The stream bed is sand, gravel, and boulders, underlaid with shale. The toe of the sloping deck joins with the upstream cut-off wall, which extends through the gravel into the shale. The downstream cut-off wall extends into the shale. Between these two cut-off walls is the natural sand, gravel and

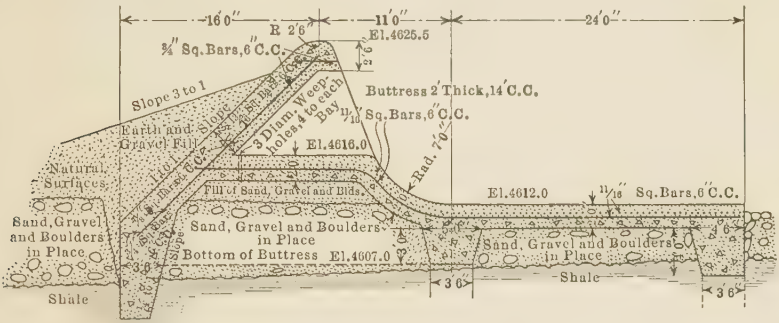


FIG. 23.—Section of Corbett diversion dam on Shoshone. Shoshone Project, Wyo.

boulder on which the floor is built. Weep holes through the floor would seem desirable to relieve the upward pressure due to any water creeping past the upstream cut-off wall; no weep holes have apparently been provided. The floor is 2 feet thick, reinforced with $1\frac{1}{16}$-inch square bars placed longitudinally and transversally and spaced 6 inches apart in both directions. The crest of the sloping deck is $13\frac{1}{2}$ feet above the surface of the floor. The deck slab is 2 feet 6 inches thick, reinforced transversally with $\frac{3}{4}$-inch square bars, 6 inches center to center, and longitudinally with $\frac{7}{8}$-inch square bars, 6 inches center to center, for the lower part and $1\frac{3}{16}$-inch square bars, 6 inches center to center, for the upper part. Against the sloping face is a filling on a 3 to 1 slope of the material excavated from the stream bed. The buttresses are 2 feet thick, spaced 14 feet apart on

centers, and extend under the floor well into the gravel. Through the abutment walls, under the deck are 12 × 12-inch holes for the passage of air, supplied through an air shaft in each abutment wall. This supply of air is necessary to replace the air carried out by the falling sheet of water. Experience has shown that at least in one case omission of this provision to let air under the falling sheet of water has resulted in producing objectionable vibrations in the body of weir. The reinforced concrete sloping face and buttresses are much heavier than found necessary from a consideration of the pressures acting on the face; this heavy design makes it doubtful that the cost of this structure is less than would have been the cost of a solid concrete weir of the same height. The dam was completed in 1907. The actual total cost to the Reclamation Service, including all overhead cost, was $127,277.

For the design of a number of reinforced concrete dams of the same type, of greater height and of lighter construction, the reader may consult Wegmann's treatise on The Design and Construction of Dams and the references accompanying this chapter.

Three Mile Falls Diversion Weir, Umatilla River, Ore.—This weir has a total length of about 820 feet, divided by triangular buttresses, 20 feet apart on centers, into 41 panels, each closed by concrete arches (Fig. 24). The axis of the dam is curved upstream on a radius of 1,200 feet. The foundation is rock; the base of the buttresses are carried into the solid rock in trenches, the bottom of which are made rough to resist the tendency to slide; along the connection of the toe of the sloping arches with the foundation a cut-off wall is carried into the rock. The maximum height from the foundation to the crest of the weir, formed by a horizontal shelf, is 24 feet. The upstream face of the arches has a slope of 1¼ to 1; this face has a radius of curvature of 18 feet. The arch ring is not reinforced; its thickness tapers from 24 inches at the base to 12 inches at the top. The buttresses are 24 inches thick, reinforced with two sets of ½-inch bars placed vertically 24 inches on centers, each set 6 inches from each face of the buttress. Through the buttresses are inspection openings, 6 feet high and 24 inches wide, connected by a reinforced gallery walk.

The maximum observed flood flow, covering the period from 1897 to 1913 was 15,200 second-feet. The maximum capacity

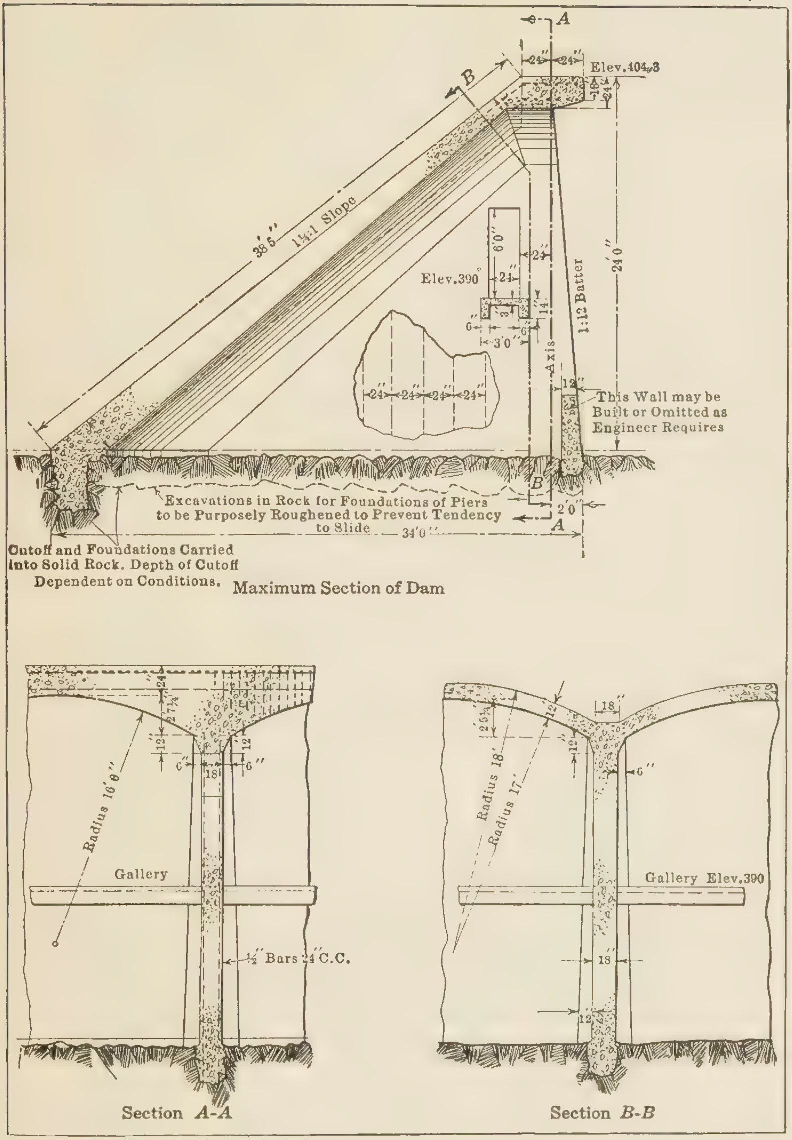


FIG. 24.—Arched deck reinforced concrete diversion weir on Umatilla River. Umatilla Project, Ore.

over the weir crest provided for is 38,000 second-feet, obtained with a depth of water of 5.7 feet on the crest.

East Park Feed Canal Diversion Weir, Orland Project, Calif.—A different type of diversion weir, of light reinforced concrete

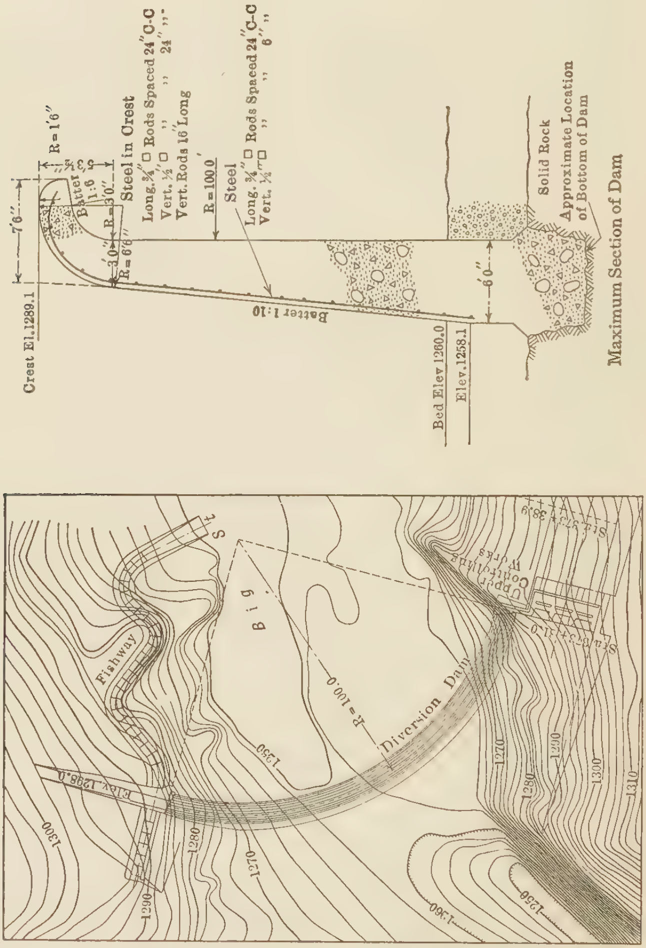


FIG. 25.—Arched diversion weir. Orland Project, Calif.

construction, is illustrated by that planned and now being constructed by the U. S. Reclamation Service for the diversion of the East Park feed canal of the Orland project, in California. This weir is a reinforced arched dam, formed by a single arch, and is designed for an estimated high water level of 7 feet above

the crest of the dam (Figs. 25A and B). The radius of curvature, measured from the downstream face of the arch ring, is 100 feet. The length measured on the arc is about 155 feet, the central angle being about 90°. The stream bed is gravel, underlaid at a small depth with rock. The base of the dam is carried to a depth of 3 to 4 feet into solid rock, and is from 4 to 10 feet below the natural stream bed. The maximum height from the stream bed to the crest is about 29 feet; the thickness of the arch at the bottom is 6 feet and tapers to a thickness of 3 feet near the crest which is formed by a curved projecting lip. The downstream face of the arch is vertical and the upstream face has a batter of 1 to 10. Vertical reinforcement of ½-inch rods, 6 inches center to center, and horizontal longitudinal reinforcement of ¾-inch rods 24 inches center to center is placed near the upstream face; this reinforcement adds security against temperature stresses. Not considering the reinforcement, the stress in the concrete may be obtained by the well-known formula for arch dams:

$$T = \frac{PR}{S}$$

where T = thickness in feet.

P = water pressure in tons per square foot, at the depth for which T is obtained.

R = radius of curvature of dam in feet.

S = pressure on the concrete in tons per square foot.

The value of S, by this formula, for the dimensions given, at a depth of 29 feet, is 13.5 tons per square foot, which is very moderate as compared with a number of higher dams of this type built in New South Wales (see references).

Open and Collapsible Weirs.—A brief general description of this class of weir and the conditions for which it is best adapted have been previously stated. One type of open weir has been presented in the description of the headworks of the Truckes Carson project; another type has been described in the discussion of wooden-frame diversion dams. In both of these types the framework and operating platform were permanent structures, which is objectionable in a stream carrying much floating material. Where a clear, unobstructed waterway is required, a large number of different types of removable or collapsible weirs have been devised. These have been largely used in the

improvement of rivers for navigation, also to a considerable extent for the crest of diversion weirs on rivers in India. Their use on irrigation projects in the United States is limited to a few cases, from which most of the examples illustrated and described below have been selected. For a more complete presentation of the various types of movable weirs or dams, the articles listed in the references and the following valuable works are of special value:

Improvement of Rivers, by Thomas & Watt (1913); Wiley & Sons, New York.

The Design and Construction of Dams, by Wegmann (1911); Wiley & Sons, New York.

The Irrigation Works of India, by Buckley; Spon, New York & London.

Moore Weir on Cache Creek, Calif. (Fig. 26).—This weir is constructed on the lower part of Cache Creek, where the stream bed is sand and gravel. The low banks of the river did not permit raising the flood flow level by a closed weir; for this reason, and also because of the floating material carried by flood flows, a removable open weir was adopted (Plate III, Fig. D). The weir is 400 feet long between abutments; it consists of the removable superstructure and of the substructure, consisting of a redwood floor 30 feet wide, nailed to rows of 4 × 12-inch sills, bolted to the top of the sheet piling cut-off wall on the upstream edge of the floor and to the tops of four parallel rows of 10 × 12-inch piling, 7 feet apart, with the piles 10 feet apart in each row. There is a double row of sills to each row of pile, with a sill on each side of the top of the piles. The sheet piling extends to a depth of 14 feet; the anchor piles increase in depth from 20 feet for the first upstream row to 24 feet, 30 feet and 36 feet for the succeeding rows. In this manner anchorage is obtained against uplift pressure, and security against sliding is provided.

The removable superstructure consists essentially of flashboard posts, tie rods and horizontal flashboards. The maximum height of flashboard crest is about 8 feet from the floor. The posts are spaced 5 feet apart. Each post is made of 6 × 6-inch timber with a groove for flashboards, formed with extra guide pieces. The lower end of the post fits into a socket, formed in a sill built of two pieces secured to the floor. The top of the post is slotted to receive the upper threaded end of the tie rod, which holds the post in place against the water pressure. The lower

end of the tie rod is linked to a pivoting iron rod, bolted to the sheet piling. The posts are brought in line by screwing the nuts, and the rods are lifted out of the slots when it is desired to remove the posts. The removal of the posts is done after the end of the irrigation season, when there is little water in the river and they are replaced after the flood flows in the spring when the stream flow level must be raised to deliver a full supply in the canal.

The diversion weir was constructed in the fall of 1903; the maximum flood flow is about 40,000 cubic feet per second; the maximum flow after April 1 does not usually exceed 5,000

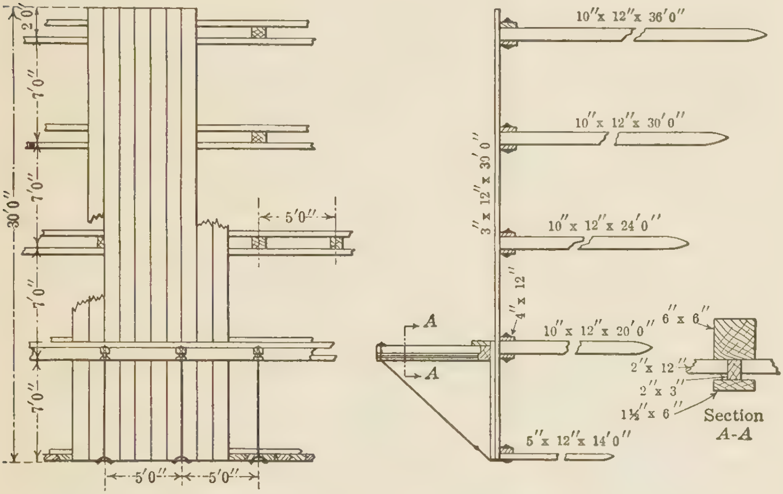


FIG. 26.—Moore diversion weir. Yolo Water & Power Co., Calif.

second-feet, and by June 1 is usually less than 1,000 second-feet.

The structure with the wooden headgate to the canal cost about $10,000; it has required practically no repairs and is in good condition 11 years after its completion. It has given good service, but would not be well adapted where the stream flow is too large to permit the erection of the removable structure after the beginning of the irrigation season.

Diversion Weir of Las Vegas Irrigation Project, New Mexico (Fig. 27).—This weir is somewhat similar in operation to that used on Cache Creek, in that the superstructure is partly formed by removable posts and flashboards, but it differs in that per-

manent concrete buttresses are also used. The structure is built on the Gallinas River, near the city of Las Vegas. The river is torrential, subject to cloudbursts, which produce sudden large flood flows, carrying logs and other floating material. To meet these conditions, a weir was designed which would have a quickly removable superstructure and which would offer little obstruction to the flow. The length of the weir between abutments is 119 feet 8 inches. The superstructure consists of permanent triangular concrete buttresses, spaced 12 feet apart on centers, and of intermediate collapsible steel posts, which with the buttresses are

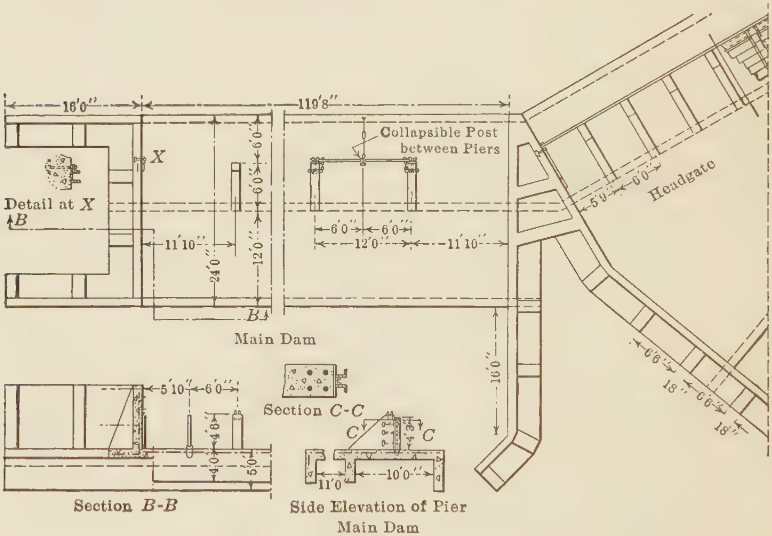


Fig. 27.—Collapsible diversion weir of Las Vegas Irrigation Project, N. M.

the supports for horizontal flashboards. The buttresses are 4 feet 6 inches high above the floor, and permit the insertion of flashboards to a maximum height of 4 feet. The upstream edge of the buttress is vertical and in the same plane as the erected collapsible posts. One end of the flashboards fits in the grooves, formed with angles placed on the edges of the buttresses, and the other end bears against the collapsible posts. The flashboards are 8 inches wide, 3 inches thick and 5 feet 9 inches long. A removable plank walk is supported on top of the buttresses; from this walk the flashboards are placed in position. Six flashboards form a set for each panel, between a buttress and the adjacent

post. The ends of the flashboards next to the buttress are chained to the buttress to prevent them from floating downstream, when the weir is opened. The bottom of each post is connected to the floor with a pivot joint and the top of the post connects with the upper end of a tie rod through a loose link joint, which can be easily disconnected from the plank walk to let the post drop and open the weir. With the posts collapsed, the flashboards and the foot walk removed, the only obstruction to the flow are the concrete buttresses, which leave clear openings between them of 11 feet.

The superstructure is formed of a reinforced concrete floor, 24 feet wide, 1 foot thick, supported on three parallel cut-off walls. The upstream cut-off wall along the upstream edge of the floor extends down to a maximum depth of 5 feet below the upper surface of the floor, which in several places brings it on solid rock. The downstream cut-off wall along the downstream edge of the floor and the intermediate cut-off wall extend to a depth of 4 feet. The floor is reinforced with two layers of No. 12 woven wire, 4-inch mesh, placed near the upper face. The path of percolation is 46 feet, which for a height of crest of 4 feet above the floor gives a ratio of nearly 12, which is evidently ample. Weep holes have not been provided and, while in this case the upward pressure is small, it would add to the stability to have them.

Diversion Weir on Murrumbidgee River, New South Wales, Australia (Plate IV, Figs. C and D).—The weir is on the valley portion of the river, where a rise in flood flow water level by a closed permanent weir had to be avoided on account of the low banks and the possibility of changing the course of the river channel. To meet these conditions, an open movable weir was constructed. The essential parts of the weir are a set of Chanoine shutters, which close a clear width of waterway of 165 feet, and two Stoney roller gates to close two adjacent sluiceway openings, each 40 feet wide, in front of the canal headgates. Except for the pier between the Stoney gates and the pier between the end of the Chanoine shutters and the adjacent Stoney gate, the entire width of waterway is unobstructed. The Chanoine shutters with the Stoney gates are to be used to raise the water level during the period of low flow. An increase in stream flow caused by freshets up to the capacity of the sluices is disposed of through the Stoney gates, and only during periods of high flow are the shutters collapsed. One of the sluiceway channels is

provided with lockgates for river navigation. The Chanoine shutters permit raising the water level to a height of 14 feet 4 inches above the floor. Each shutter (Fig. 28) is 2 feet 11½ inches wide, and 13 feet 10 inches high, and is pivoted by cast-iron bearings on a horizontal shaft near the center of the panel at

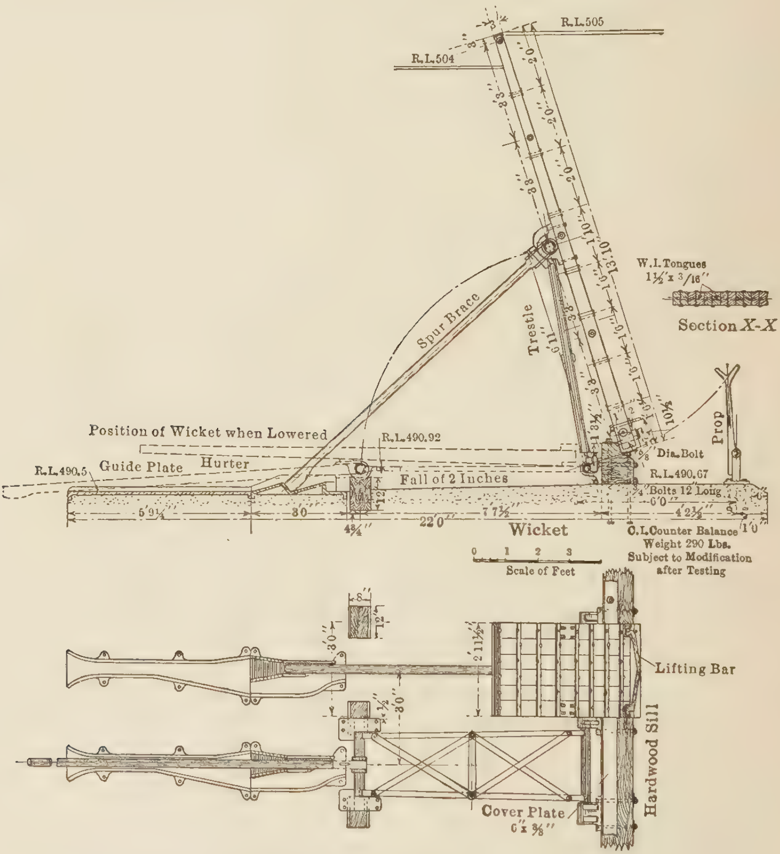


Fig. 28.—Details of Chanoine shutters used on diversion weir on Murrumbidgee River. New South Wales, Australia.

such a height that, as long as the stream flow is sufficient to bring the downstream water level to a height of about 1 foot below the crest of the shutter, the shutter will automatically tilt open to a limited position, fixed by a lug on the castings. For these conditions of stream flow a full head can be diverted in the canal without having the shutters closed. Each shutter is supported

Fig. A.—Same as Pl. IV, Figs. C and D, with shutters raised to leave waterway partly open.

Fig. B.—Collapsible diversion weir of Prewitt Reservoir Project, Colo.

(*Facing page 74*)

on a collapsible A-frame, formed by a trestle, held upright by an inclined prop. The bottom piece of the trestle is a horizontal shaft connected to cast-iron pivot bearings, bolted to the wooden floor sill, anchored by bolts to the concrete. The top piece of the trestle is also a shaft, to which are connected the pivot bearings of the shutter and the pivot bearing of the supporting prop. The lower end of the prop fits into a special slot, with guide channel, called a hurter, specially designed to engage and disengage the foot of the prop.

During flood times the shutters are lowered and lay flat on the floor. When the low water level makes it necessary to raise the shutters, the lifting bar at the foot of the shutter is caught with a hooked rod and by pulling on this rod the shutter is gradually lifted until the lower end of the prop drops in the slot of the hurter; the lifting bar is then released and the water pressure on the shutter brings it to its erected position. When the shutters are being raised, the flow is concentrated through the opening where the shutters are down; to avoid the difficulty of lifting these last shutters in the strong current, all the shutters or part of them may be left partly opened by allowing the lower end of the shutter to drop in the fork of the hinged prop (Plate V, Fig. A). To finally close the openings, the props are pulled out and the shutters take the erected position.

To lower the shutter, the lifting bar at the bottom of the shutter is caught with the hooked rod and on pulling, the shutter first revolves to a nearly horizontal position, fixed by the lug on the pivot castings; the pull is continued until the lower end of the inclined strut is pulled out of the notch of the hurter; the lifting bar is then released, and the shutter drops down in its lowered position. The lifting hook is operated from a small flat-bottom boat, on which is a winch with wire rope. The boat is connected by a rope and a pulley to a wire rope stretched across the river, parallel to the weir, about 60 feet upstream.

The open space between the side of the shutters is about ½ inch, which is about the minimum that can be used. Where the leakage through these openings is objectionable, the openings could be closed by lining the edges with rope, which the water would press into the joints.

The substructure consists of a concrete floor, 24 feet 6 inches wide and 3 feet thick, built across the river, on solid granite for part of the way and on foundation cross walls where the depth

to rock deepens. The cross walls, placed at right angles to the length of the floor, are 3 feet thick and 7 feet apart. The sand foundation in between is confined by an upstream and a downstream toe wall, along the edges of the floor which extend into the solid rock. The floor for this part is reinforced with steel rails.

Diversion Weir of Prewitt Reservoir Project, Colorado (Plate V, Fig. B).—This weir was built on the South Platte River at a point where an entirely closed weir could not be used

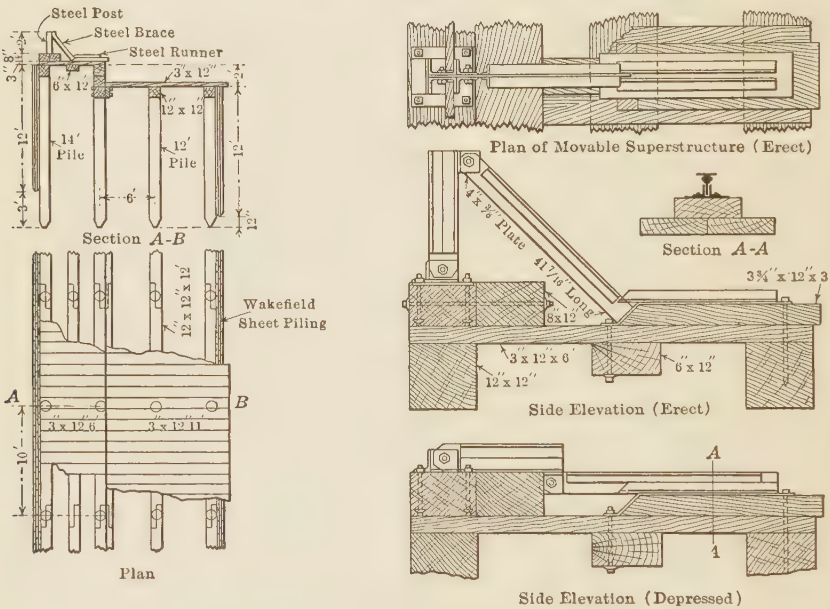


Fig. 29.—Details of movable superstructure collapsible diversion weir Prewitt Reservoir Project, Colo.

on account of adjacent lowlands, which would be flooded during flood flows. To force a full supply in the canal it was necessary to raise the water level in the river to a height of 4½ feet above the stream bed; this was done by constructing a permanent substructure extending 2½ feet above the stream bed, on which was built a removable flashboard crest to raise the water level 2 additional feet (Fig. 29). At ordinary stages the depth of water over the permanent weir crest is 1 to 2 feet, frequently dropping to 6 inches. During high water the river channel spreads from the normal channel width of 500 feet to 1,500 feet. The weir is

about 500 feet long and the remainder of the flood flow channel is closed by an earth embankment, with a cut-off wall of Wakefield sheet piling, extending 10 feet into the soil and projecting 4 feet into the embankment. Against the downstream side of the sheet piling is a row of round piles, 10 feet on centers. The upstream face of the embankment is lined with reinforced concrete slabs, 4 inches thick.

The substructure of the weir consists of a stepped wooden floor nailed to heavy sills of 12 × 12-inch timber, supported on four rows of round piles, and of sheet piling cut-off walls along the upstream and downstream edge of the floor. On the upstream part of the floor, which is 2 feet above the downstream part of the floor, are built the collapsible frames which support the flashboards. The frames are spaced 4 feet on centers. Each frame consists of a post with grooves to receive the flashboards, and of an inclined brace. The post, which is vertical when erected, is connected at the bottom to the floor through a pivot bearing and at the top to the upper end of the brace through a second pivot joint. The lower end of the brace slides in a slot or runner, and when erected bears on a shoulder formed at the upstream end of the slot. The post, brace and runner are made of structural steel. The flashboards are 1½ × 10 inches and 3 feet 10 inches long.

Two sluiceways have been provided: one at the end of the weir adjacent to the canal headgates to maintain a clear channel in front of the headgates, and the other at about the center of the weir to direct the flow around an island toward the intake of another canal. Each sluiceway has six openings, each 5 feet wide, separated by permanent steel frames and controlled by flashboards. The floor of the sluiceway is a continuation of and on the same level as the lower or apron floor. Each frame is made of ¼-inch steel plate, stiffened with angles, with flashboard grooves formed by the legs of two angles. The height of the frame is 7 feet; its upstream edge is vertical; its width at the bottom is about 6 feet 6 inches and at the top 24 inches. The top of the frames support a 2 × 12-inch foot plank.

The operation of the weir requires that the posts and flashboards be placed or removed at periods of low flow by wading in the water.

Diversion Weir of Crocker-Huffman System on Merced River, Calif.—This weir is of special interest because of the collapsible

steel structure which forms the upper part of the main portion of the diversion weir. The total length of the weir between abutments is about 700 feet; for 155 feet adjacent to one abutment and 60 feet adjacent to the other abutment the weir cross section is the regular Ogee gravity type, built of concrete on a hardpan foundation. The remainder of the weir, nearly 500 feet in length, has the same Ogee cross section, from the base up to 4 feet from the crest, the upper 4 feet being formed by the steel collapsible structure (Fig. 30). This type of collapsible structure used was invented by J. C. Wheelon, Chief Engineer

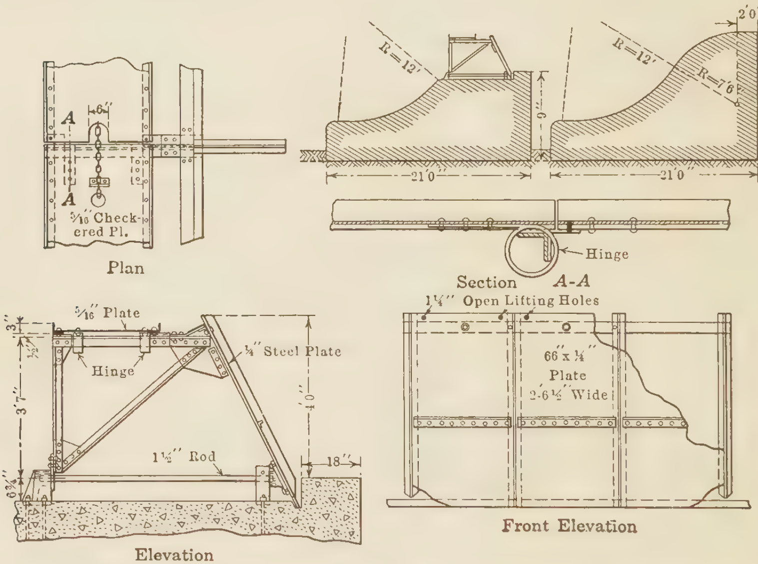


FIG. 30.—Details of collapsible gates on diversion weir of Crocker-Huffman system. Merced, Calif.

of the Bear River Canal Co. in Utah, who first used it on the Bear River near Garland, Utah, to replace the upper 5 feet of the old crib weir, in order to reduce the pressure on the old weir during the flood flow when the collapsible crest would be folded down. The upper 5 feet of the crib was removed and the collapsible structure bolted to the timbers. The design used on the Merced River diversion weir was almost identically the same, varying only in minor details.

The collapsible structure is formed of trapezoidal steel frames, a runway built in sections supported on top of the frames, and

steel-plate flashboards or gates supported on the upstream edge of the steel frames. The flashboards are removable and are carried on cars which travel on the runway. The frames and the runway are designed to fold over each other and lay flat on the masonry crest. The frames are spaced 8 feet apart and revolve at their base, the lower member of the frame being a shaft which rests on cast-iron bearings bolted to the concrete. The runway is built in sections 8 feet long; one end of each section is hinged to the top angle member of the frame; the other end is connected to the top member of the next frame when the framework is erected, but disconnected from it when the frames are to be collapsed and is then free except for a loose chain connection with the hinged end of the adjacent section, so that when a frame and its hinged runway section is lifted it brings with it one end of the chain with which the next frame and runway section are lifted. With these two hinged joints, each frame and the section of runway hinged to it may be folded to lay flat on the masonry crest. When the frames are erected, a longitudinal brace connects the upper ends of the upstream members of the frames, and the 8-foot wide opening between frames is divided into three smaller openings by two intermediate flashboard supports. The flashboards are made of ¼-inch plate, 5 feet 6 inches long and 2 feet 6½ inches wide, stiffened by an angle riveted a short distance below the upper half of the plate.

The operations required to erect or lower the movable structure are evident from the illustration and the descriptions given above. Depending on the character of the stream flow, there may be difficulties in operation. For instance, a sudden rise in the stream flow, occurring when the structure is erected, may cause the overtopping of the foot walk and make the removal of the flashboards and the lowering of the bents impossible. Mr. Wheelon, the inventor of this structure, states that these conditions have occurred on two or three occasions. These stream flow conditions are, however, rather unusual, and the difficulties resulting would affect also some of the other types of collapsible weirs.

REFERENCES FOR CHAPTER I

Flow Over Diversion Weirs:

Bazin, H.—Experiences Nouvelles Sur l'ecoulement en Diversoir—6me art. —Annales des Ponts and Chaussees—Memoires et Documents—1898—2me trimestre.

RAFTER, GEORGE W.—On the Flow of Water Over Dams—Trans. Am. Soc. Civ. Eng.—Vol. XLIV, 1900.

The Computation of Height of Backwater Above Dams—p. 454 Engineering News—Nov. 1, 1906.

HORTON, R. E.—Weir Experiments, Coefficients and Formulas—Water Supply Paper No. 200, U. S. Geological Survey—1907.

PFAEHLER, RICHARD—The Spillway Dam—p. 83 Engineering Record—Jan. 16, 1909.

STEVENS, J. C.—A Comparison of Stream Discharges Indicated by Current Meter and by Weir Formulas, Yakima River, Wash.—p. 481 Engineering News—April 28, 1915.

MARTIN, W. F.—Discharge Measurements and Formulas for Some Large Overflow Dams—p. 321 Engineering News—Sept. 29, 1910.

MOGENSON, P.—Formulas for Discharge of Overfall Dams—p. 573 Engineering News—Nov. 24, 1910.

WILLIAMS, G. S.—The Flow of Water Over Dams—p. 38 Engineering News—Jan. 12, 1911.

WEIRS ON PERVIOUS FOUNDATIONS:

CLIBBORN, L. J.—Experiments on the Passage of Water Through Sand—Technical Paper No. 97, Roorkee, India—1902.

Grouted Rubble Core Walls for the Weirs of the Delta, Barrage, Egypt—p. 147 Engineering News—Feb. 9, 1905.

The Laguna Dam—p. 146 Engineering News—Feb. 9, 1905.

VINCENT, E. D.—The Construction of the Laguna Dam, Colorado River—p. 213 Engineering News—Feb. 27, 1908.

Upward Pressure in Masonry Dams—p. 84 Engineering Record—July 18, 1908; p. 167 Engineering Record—Aug. 8, 1909.

VINCENT, E. D.—Turning the Colorado River and Completing the Laguna Dam—p. 615 Engineering News—June 10, 1909.

BLIGH, W. G.—The Practical Design of Irrigation Works—D. Van Nostrand Co., New York—1910.

Failure of Canal Headworks, Egypt—p. 487 Engineering Record—Apr. 9, 1910.

BLIGH, W. G.—Weirs on Porous Foundations and with Pervious Floors—p. 444 Engineering News; Apr. 13, 1911—p. 708 Engineering News—Dec. 29, 1910; p. 52 Engineering News—Jan. 12, 1911.

BLIGH, W. G.—Sheet Piles as a Means of Decreasing Permeability of Porous Foundations—p. 109 Engineering News—Jan. 26, 1911.

KOENIG, A. C.—Dams on Sand Foundation—p. 290 Engineering and Contracting—March 8, 1911.

The Under Pressure of Water on Dams—p. 527 Engineering and Contracting—May 3, 1911.

KOENIG, A. C.—Dams on Sand Foundation—Trans. Am. Soc. Civ. Eng.—Vol. LXXIII, Sept., 1911.

WEGGMANN, E.—The Design of Masonry Dams, Should Hydrostatic Pressure Underneath and Ice Pressure be Included—p. 594 Engineering News—Nov. 16, 1911.

GODFREY, E.—Hydrostatic Pressure Beneath Dams—p. 661 Engineering News—Nov. 30, 1911.

Harrisson, C. L.—Provision for Uplift and Ice Pressure in Designing Masonry Dams—p. 663 Engineering News—Nov. 30, 1911.

Bligh, W. G.—Lessons from the Failure of a Weir and Sluices on Porous Foundations—p. 266 Engineering News—Feb. 6, 1913.

Harris, A. L.—Overflow Dams in Service Five Years—p. 541 Engineering Record—Nov. 15, 1913.

Harris, A. L.—The Diversion of Irrigating Water from Arizona Streams—Trans. Am. Soc. Civ. Eng.—Vol. LXXII, Dec., 1914.

Timber Crib Dams:

Construction Repairs and Subsequent Partial Destruction of Arizona Canal Dam, near Phoenix, Ariz.—p. 450 Engineering News—Apr. 27, 1905.

Temporary Crib Dam, Shoshone Project—p. 545 Engineering News—Nov. 23, 1905.

A Successful Low Timber Dam for Broad Sandy River Channels—p. 645 Engineering Record—Dec. 5, 1908.

The Diversion Dam Across the Rock River, Illinois—p. 138 Engineering News—Aug. 5, 1909.

The Construction of the Rainbow Falls Hydroelectric Development—p. 292 Engineeering Record—March 12, 1910.

Cole, D. W.—The Shoshone Dam of the U. S. Reclamation Service—p. 88 Engineering Record—July, 23, 1910.

Dam of Big Bend Power Project, Feather River—p. 422 Engineering and Contracting—October 16, 1912.

Methods and Cost of Rebuilding South Dam, Head of Rock River Rapids at Lock 35 of the Illinois and Mississippi Canal—p. 555 Engineering and Contracting—Nov. 13, 1912.

Timber Dams and Rock-fill Dams, Part II—Bull. 249, Office of Experiment Stations, U. S. Dept. of Agriculture, Washington, D. C.—1912.

Wooden-frame and Steel-frame Dams:

Steel Dam for Reservoir in Arizona—p. 148 The Engineer—Aug. 12, 1898.

Diversion Weir: The Kern River Co.'s Hydroelectric Power Plant—p. 56 Engineering News—July 21, 1904.

Pueblo Rocky Ford Irrigation Project—p. 141 Engineering Record—July 29, 1911.

Hammatt, W. C.—Three Approved Designs for Weir Dams for Irrigation Canals—p. 646 Engineering and Contracting—June 5, 1912.

Hammatt, W. C.—A Western Type of Movable Weir Dam—p. 121 Trans. Am. Soc. Civ. Eng.—Vol. LXXVI, Dec., 1913.

Gravity Concrete Diversion Weirs:

Stannard, J. D.—The Granite Reef Diversion Dam—p. 367 Engineering News—Oct. 1, 1908.

Development of a Practical Type of Concrete Spillway Dam—p. 461 Engineering Record—October 24, 1908.

Teichman, F.—The Granite Reef Dam and Gate Mechanism—p. 1 Engineering News—Jan. 7, 1909.

The Granite Reef Diversion Dam, Arizona—p. 561 Engineering Record—May 20, 1911.

Distribution System of Pueblo Rocky Ford Irrigation Project—p. 602 Engineering Project—Nov. 18, 1911.

The Irrigation System of the Southern Alberta Land Co.—p. 420 Engineering Contracting—Apr. 10, 1912; p. 589 Engineering Record—May 27, 1911.

Thomas, D. G.—New Water Supply Intake at Denver—Engineering Record—Oct. 19, 1912.

Harris, A. L.—Overflow Dam in Service Five Years—p. 541 Engineering Record—Nov. 15, 1913.

Harris, A. L.—The Diversion of Irrigating Water from Arizona Streams—Proc. Am. Soc. Civ. Eng.—Jan., 1914.

Stevens, J. C.—Seros Hydroelectric Development in Spain, Part I—p. 250 Engineering Record—Aug. 29, 1914.

Stevens, J. C.—Seros Power Development, Lerida, Spain: Diversion Dam and Canals—p. 473 Engineering News—Sept. 3, 1914.

Reinforced Concrete and Arched Diversion Weirs:

Beardsley, R. C.—The Design of Buttressed Dams of Reinforced Concrete—p. 452 Engineering News—Apr. 23, 1908.

The Cost of the Corbett Diversion Dam, Shoshone Irrigation Project—p. 219 Engineering Record—Aug. 22, 1908; p. 126 Engineering Contracting—Aug. 26, 1908.

The La Prele Dam—p. 373 Engineering Record—April 3, 1909.

Hume Lake Dam—Journal of Electricity, Power and Gas—Oct. 30, 1909.

The Failure of a Small Concrete Dam, Dansville, N. Y.—p. 24 Engineering Record—Jan. 1, 1910.

Reinforced Concrete Deck Dam, Pulaski, Va.—p. 157 Engineering Record—August 6, 1910.

Beardsley, R. C.—A Criticism of a Cellular Dam Design and a Discussion of Reversed Dam—p. 139 Engineering Contracting—Feb. 1, 1911.

Spillway Dam for the Lost River Diversion Channel (Klamath)—p. 311 Engineering Record—March 18, 1911.

Godfrey, E.—A Comment on Reversed Dams—p. 293 Engineering and Contracting—March 8, 1911.

Beardsley, R. C.—Reversed Dams—p. 348 Engineering and Contracting—March 22, 1911.

Wade, L. A. B.—Concrete and Masonry Dam Construction in New South Wales—Vol. CLXXVIII, p. 1, Proc. Inst. C. E.

Fuich, J. K., and Thoman, W. F.—Reinforced Concrete Hollow Dam of the Buttress Type—Canadian Engineer.—May 29, 1913.

Eastwood, J. S.—The Ultimate Dam—Western Engineering—Sept., 1913.

Eastwood, J. S.—The New Big Bear Valley Dam—Western Engineering—Dec., 1913.

The Design and Construction of the Bassano Dam, Part I—p. 425 Engineering News—Aug. 27, 1914; Part II—p. 484 Engineering News—Sept. 3, 1914.

Collapsible Diversion Weirs:

Thomas, B. F.—Movable Dams—p. 71 Proc. Am. Soc. Civ. Eng.—Feb., 1898.

Bear Trap Dam for Regulating Works, Chicago Drainage Canal—Engineering News—March 24 and May 26, 1898.
A New Automatic Movable Dam—p. 222 Engineering Record—March 8, 1902.
The Reversed Parker Bear Trap Gate (Reconstruction of the Lake Winnibigoshish Dam)—p. 250 Engineering Record—Sept. 13, 1902.
Wilson, H. M.—Automatic Drop Shutters for the Betwa Irrigation Dam, India—p. 494 Engineering News—June 4, 1903.
Rolling Dams at Schweinfurt, Bavaria—p. 57 Engineering News—Jan. 19, 1905.
Movable Dam and Lock of the Rice Irrigation and Improvement Association, Mermenton River—p. 31 Engineering News—Sept. 28, 1905.
Wheelon, J. C.—A Collapsible Steel Dam Crest, Bear River, near Garland, Utah—p. 353 Engineering News—Oct. 3, 1907.
The Permanent Improvement of the Ohio River (Poiree Dam), Vol. II—p. 13 Engineering Record—Jan. 2, 1909.
The Permanent Improvement of the Ohio River (Chanoine Dam), Vol. III—p. 41 Engineering Record—Jan. 9, 1909.
The Permanent Improvement of the Ohio River (Bear Trap), Vol. IV—p. 69 Engineering Record—Jan. 16, 1909.
The Permanent Improvement of the Ohio River (Frame Dam), Vol. V—p. 97 Engineering Record—Jan. 23, 1909.
The Ohio River Dam and Lock No. 37, near Cincinnati, Ohio (Chanoine Wickets and Bear Traps)—p. 235 Engineering News—March 4, 1909.
The Government Dam Across the Rock River at Sterling, Illinois (Taintor Gates)—p. 138 Engineering News—Aug. 5, 1909.
Wade, L. A. B.—Irrigation from the Murrumbidgee River, New South Wales—Paper read before the Sydney University Engineering Society—Nov. 17, 1909—William Applegate Gullick, Govt. Printers, Sydney, Australia.
Bear Trap Dam, Glenwood Hydroelectric Plant, Central Colorado Power Co.—p. 807 Engineering Record—June 25, 1910.
Rolling Dams for Irrigation in Mexico—p. 302 Engineering Record—Sept. 10, 1910.
The Las Vegas Irrigation Project, New Mexico—p. 396 Engineering Record—Oct. 8, 1910.
The Use of the Rolling Dam in Europe—p. 492 Engineering Record—Oct. 29, 1910.
Rio Guaso Irrigation Dam with Automatic Shutters (Cuba)—p. 523 Engineering Record—May 13, 1911.
Stickney, C. F.—An Automatic Dam Crest—p. 296 Engineering News—Feb. 15, 1912.
The Movable Diversion Weir at Berembed, New South Wales—p. 486 Engineering Record—May 4, 1912.
Prewitt Reservoir Project—Engineering Record—Dec. 28, 1912.
Ulrich, J. C.—The Prewitt Reservoir Proposition—p. 96 Trans. Am. Soc. Civ. Eng.—Dec., 1914.

CHAPTER II

SCOURING SLUICES, FISH LADDERS, LOGWAYS

A scouring sluice is a general term used to denote either an open sluiceway or an undersluice. An open sluiceway is an open bay or panel in the diversion weir extending from the floor of the sluiceway, usually placed level with or lower than the floor of the weir, up to the crest of the weir. An undersluice is an opening through the body of the weir and does not extend to the crest of the weir; its use is usually limited to comparatively high diversion weirs (generally over 10 feet) and where the amount of silt and sand carried by the stream is not large.

Purpose.—The main objects of a scouring sluice are:

First.—To maintain a well-defined channel in front of the head-gates to the canal by scouring the silt or sand deposited in front of the gates.

Second.—To prevent the entrance into the canal of the coarse material carried by the river water.

Third.—To regulate the river water level, within certain limits of minor stream flow variations. This object is only of real value where the crest of the weir is provided with a collapsible or removable superstructure and when this method of regulation is more convenient than the adjustment of the collapsible crest.

Necessity for; Method of Operation and Efficiency.—The necessity for scouring sluices will depend largely on the amount of material carried by the stream flow and the character of the stream and stream bed. In the United States the majority of streams used for irrigation carry only a small amount of sediment; there are some streams, however, especially in Arizona, New Mexico, and Texas, which carry large volumes of material. In India and Egypt silt problems of great difficulty have had to be contended with in the working out of satisfactory plans for the diversion works, and the lessons derived from the results obtained are of much value where similar difficulties may arise.

A better understanding of the necessity for scouring sluices is obtained by first considering the difficulties to be overcome.

The construction of a closed diversion weir across a river, except on a river carrying little or no sediment, will result in the deposition of material on the upstream side of the weir, and where the river bed is anything different from solid rock there is a gradual movement of the stream bed material. These combined actions have a tendency to raise the river bed on the upstream side up to the crest of the weir, and in the case of stream beds of coarse gravel or heavier material the erosive effect of flood flows is

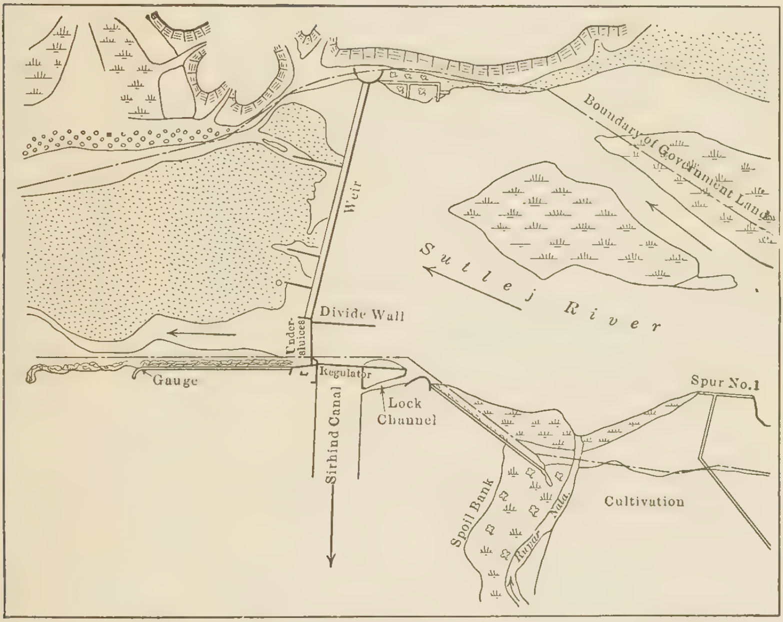


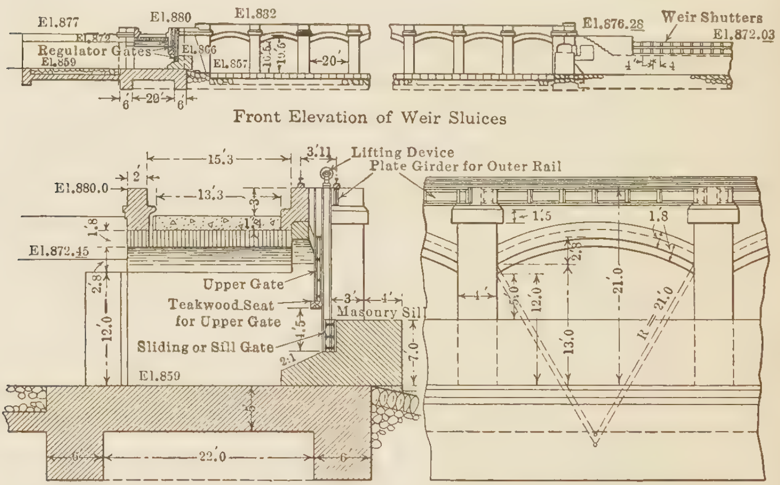
FIG. 31.—General layout for diversion works for Sirhind Canal. Sutlej river, Punjab, India.

not always sufficient to keep the stream bed below or at the crest level.

In many cases on Indian rivers the use of large scouring sluices located not only at both ends of the weir in front of the canal headgates but also in the center of the weir, has not been able to prevent these effects; with the result that islands have built up on the upstream side of the weir, irregular cross channels have been formed, and the difficulties of diversion have been very great. In some cases it has been necessary to raise the weir crest with a collapsible crest. These effects have not been

entirely confined to Indian rivers, for at least in one case, observed by the writer on the Umatilla River in Oregon, a closed weir, through which, however, no scouring sluices were provided, produced sand and gravel islands which required the excavation of a channel to the headgates and the raising of the crest with sacks filled with sand. The results obtained in India have led to the conclusion that the only satisfactory way to prevent these difficulties is the use of collapsible or removable crest on a low permanent weir wall, or to make an entirely open weir.

Experience on at least one river in India, the Sutlej River, at the diversion works for the Sirhind Canal (Fig. 31), indicates



Sectional Elevation of Regulator Gates Front Elevation of Regulator Gates

FIG. 32.—Headworks of Sirhind Main Canal. Punjab, India.

that the efficiency of scouring sluices will depend largely on the method of operation. The weir, 2,400 feet long, is of the Indian rock-fill type, with a 3 to 1 upstream slope and a 15 to 1 downstream slope. The crest wall originally raised the water level to a height of 8 feet above the river bed, but it was later found necessary to raise it with collapsible shutters or gates an additional height of 6 feet. The weir is not at right angles to the stream, but slopes upstream from the end adjacent to the headgates at an angle of 15°. The headgates are parallel to the direction of the stream, and the sluiceway is directly in front of the headgates (Fig. 32). A division wall parallel to the head-

gates, 296 feet away from them, forms the sluiceway channel, divided by piers, 5.09 feet thick, into 12 openings each 20 feet wide and each regulated with a set of three gates; a bottom, a middle, and a top gate, for which separate grooves are provided.

The canal headgates consist of 13 openings, each 21 feet wide (Fig. 32). The sill of the gates was originally 2 feet above the sluiceway floor, later it was raised an additional 7 feet by building up the permanent masonry to that height, and the old gate system was changed by using a set of overpour rising steel gates to form a movable sill, over which the top water from the river passes into the canal. These gates are 3 feet 6 inches high, and when entirely lowered are housed on the downstream side of the masonry sill. To complete the closure of the gate openings above the raised position of the movable sill, another set of gates 6.25 feet high is provided.

The crest of the collapsible shutters of the weir is 15 feet above the sluiceway floor and 6 feet above the crest of the permanent sill of the headgates.

The construction of the division wall to form the sluiceway channel in front of the gates, the raising of the sill of the canal headgates, and the change in the system of gates, was done in 1893–94, after it had been found necessary to prevent the large deposit of heavier silt and sand carried through the canal headgates, which threatened to completely silt up the head of the canal. It was also decided to increase the capacity of the wasteway or escape at the twelfth mile on the main canal, in order to obtain a greater scouring effect on the deposited silt.

After these changes were made, many experiments were made on the silt problems involved and different methods and time of operation of the sluices and of the collapsible crest were tried. To better understand the results obtained, a knowledge of the character of the stream flow and of the silt is desirable. The Sutlej River has a maximum flood discharge of about 135,000 second-feet and a minimum discharge of about 3,000 second-feet. The heaviest floods usually occur during July and August. The full supply capacity of the canal at the head is 6,200 second-feet, decreased to 6,000 second-feet at a point 26 miles down.

From 1893 to 1897 measurements were made of the total amount of sediment carried in suspension in the water entering the canal. The proportion of silt to water by volume ranged from zero up to a maximum of about $\frac{1}{300}$. Most of this

sediment in suspension was fine clay or ooze, which takes considerable time to settle; part of it, probably not more than $\frac{1}{3}$, was sand. The sandy sediment in suspension with the coarser sand and material which rolls along the bottom, but which was not included in the measurement, was the main cause of the troublesome deposits in the canal.

In 1898 a special device was devised to separate the fine silt from the coarser sandy sediment carried in suspension. The results of measurement show that the average proportion of sandy sediment to water entering the canal during the flood period was $\frac{1}{1700}$; while the proportion carried off by the canal water at a point 26 miles down from the head averaged $\frac{1}{3300}$, showing that only half of the sandy silt was carried through the canal, the other being deposited in the canal. The flood period is usually in July and August; during this period the water is overladen with silt. The river water begins to clear in September, and when taken in the canal is then able to pick up and carry in suspension some of the sediment and finer sand deposited on the bed during the preceding flood flow period, thus reducing the amount of silt deposits. As the finer material is thus transported, this leaves the coarser material, of which a smaller proportion can be carried in suspension, so that in November or December the proportion of sediment carried has decreased from $\frac{1}{3300}$ to $\frac{1}{9000}$. In the above results the coarser material rolled along the bottom has not been considered, because of no satisfactory method of measurement to obtain this material.

A further study of the character of the sediment carried in suspension was made by means of a special device or sand separator, which permitted the classification of the different grades of silt, on the basis of the rate at which silt will fall through water; such that when measured in feet per second, grades of silt falling through water at the rate of 0.10, 0.20, 0.30 feet per second would be denoted as grades $\frac{1}{0.10}$, $\frac{1}{0.20}$, $\frac{1}{0.30}$, respectively. The results obtained show that silts of grade $\frac{1}{0.20}$ or less are found in as large quantity in the water of the distributaries or lateral as in the water in the main canal, indicating that only the coarser sediment above grade $\frac{1}{0.20}$ was permanently deposited in the main canal.

The object of the construction of the division wall forming the sluiceway channel in front of the canal headgates was to form a

silt basin in which the coarser material would be deposited and then scoured out by opening the sluicegates, and to concentrate and confine the scouring effect to the channel formed by this wall. To obtain satisfactory results, it was necessary to maintain the water in this channel or basin moving at a comparatively small velocity. This required that the sluicegates be kept usually closed and only opened at intervals to scour out the deposited material, and that the excess river flow be regulated by operation of the collapsible shutters on the weir. The importance of this system of operation was evidently not realized until 1900. Before this time the sluicegates were more or less open, and the water passing through the sluiceway channel would have a comparatively high velocity, carrying much silt in suspension. In this disturbed condition the cross currents or eddies were able to produce upward currents of sufficient strength to carry the coarsest sand, which in this case was mostly not greater than the $\frac{1}{0.40}$ grade, over the raised sill of the headgates into the canal. Since 1900 the correct system of operation has been enforced. The surplus river water has been discharged at the far end of the weir, through the opening formed by letting down some of the collapsible gates. This procedure produced a wide channel with comparatively low velocity, extending from the far end of the weir to the sluiceway channel, with the resulting tendency to free the water from the coarser sediment.

This method of operation depends for its success on having a sufficiently large flood flow to scour out the material deposited above the weir. On this project very satisfactory results have been obtained. The yearly deposits of silts in the canals have decreased from an average of about 15,000,000 cubic feet prior to 1900, to 5,000,000 cubic feet since 1900 up to 1903. The results of this system of operation and of the silt measurements on the canal indicate the following procedure:

First.—When possible keep the canal headgates closed during the period of flood flows when the maximum amount of silt is carried.

Second.—Keep the sluiceway gates closed as long as possible when the headgates are open, and when necessary to open only part of the sluiceway gates open the gates farthest away from the headgates, in order to concentrate the scouring channel a short distance away from the headgates.

Third.—When necessary to scour out the material deposited in the sluiceway channel, close completely the canal headgates.

Fourth.—Use a raised sill for the canal headgates and overpour gates to permit the skimming of the top water.

Fifth.—To diminish the permanent silt deposits in the canals, run as large an excess of clear water in the canals as obtainable or feasible.

It is especially desirable that the headgates be kept closed during the period when the percentage of silt carried in suspension by the river water is greater than the percentage which may be carried by the canal water; this depends on the velocities in the canal system. The velocities in the canal system should be adjusted to be not less than the critical velocities, as obtained by the formula based on Kennedy's silt theory, where $V_0 = Cd^m$, see Vol. II, Chapter IV. When the canal velocities are equal to the critical velocities and the river velocity greater than the critical velocity corresponding to the depth of water in the river, then the percentage of silt carried in the canal from the river will be too great and a deposit will occur.

The method of operation described above is based on the experience obtained on a river carrying no material coarser than sand. When the river carries heavier material, such as gravel or cobbles, then the closure of the scouring sluices and the regulation of the stream flow at the far end of the weir would result in a deposit of this material, which would probably not be scoured out by the flood flows or by the velocity created in the sluiceway channel by opening the scouring sluices. Where these conditions are obtained, a raised sill and an open or collapsible weir are probably the best solution.

DESIGN OF SCOURING SLUICES

Position.—The object of scouring sluices, as previously stated, requires that they be placed at the end of the weir adjacent to the canal headgates and that the sluicegates be at right angles to the plane of the headgates, in order that the sluiceway channel be formed directly in front of the gates. Where the river flow is diverted into two canals, one on each side of the river, scouring sluices at each end of the weir must be provided. Scouring sluices in the center of the weir are not to be recommended; they are inconvenient to operate and their effect is of little or no value.

Parts of Scouring Sluices.—Where the amount of silt carried by the river water is too small to cause troublesome silt deposits in the canal, the scouring sluices usually consist of one or more comparatively small openings through the body of the weir, closed with suitable gates, or they may be entirely omitted. Small scouring openings are of doubtful value, for while a high velocity through them may be obtainable, this velocity has only a small effect on the large body of water upstream of the opening. Where the amount of silt is considerable, a sluiceway channel must be formed in front of the headgates, extending through the diversion weir; this sluiceway channel is formed by a division wall parallel to the plane of the canal headgates with its crest on the same level as the weir crest, or preferably as high as the high water level, with a floor in between, and is divided into an upstream channel and a downstream channel by the sluicegates, placed usually in the axis of the dam. The sluiceway channel at the gates is usually divided into a number of gate openings by piers, which give the support to the gates and on top of which is built the operating platform. However, the piers may be omitted by using certain types of collapsible gates, and the operating platform may be made removable.

Capacity of Scouring Sluices.—The capacity will depend on the amount and character of silt carried by the river. The practice in India will serve as a useful guide where the river water carries considerable silt. The practice varies considerably. Existing examples in India, according to R. B. Buckley, would seem to indicate that the cross-sectional area of sluicegate openings below the weir crest should be not less than double that of the canal headgate openings, but in some successful works the area of gate openings in the sluices and canal headgate is about the same. A number of examples of Indian works, tabulated by R. B. Buckley, indicate that the area of scouring sluice openings varies from about $1/5$ to $1/20$ of the total cross-sectional area represented by the weir obstruction; most of the examples give a proportion of about $1/10$. Empirical rules which may be derived from such examples may indicate average practice in India, but even for conditions obtained there, such rules can be only of very doubtful value, for the efficiency of scouring sluices depends not only on the area of sluice openings but also on other factors, especially the relative position of the sill of the sluice openings with respect to the crest of the dam. To determine the required

capacity, it is best to consider carefully the method of operation of the sluices and to study the velocities obtained in the sluiceway channel, especially when the scouring gates are closed and the canal headgates opened, also when the scouring gates are opened and canal headgates closed. The capacity should be at least equal to the normal stream flow during the irrigation season, and preferably sufficiently larger to use the minor normal flood flows for scouring out deposits.

The velocity of the water through the head of the sluiceway channel, when diverting water into the canal headgates, with the sluicegates shut, must not be greater than either the velocity in the river or the velocity in the canal; otherwise the heavier sediment will not be deposited in the sluiceway channel, but will be carried through the canal headgates. To fulfill this condition, the cross-sectional area of the sluiceway basin formed by the division wall in front of the headgates must, for all stages of the water level, be at least larger than the cross-sectional area of the canal. With this condition fulfilled, the capacity through the scouring sluices with the sluicegates opened must be sufficiently large to produce a scouring velocity, not only at the gate openings, but also through the entire sluiceway, sufficiently high to scour out the deposited material.

The discharge through the gate openings will usually be that of an orifice with suppressed or incomplete contraction on the bottom and sides. At times the openings are entirely submerged and the discharge will depend on the difference in elevation of the water levels on the upstream and downstream sides. The coefficient of discharge in the approximate formula $Q = CA\sqrt{2gh}$, where C is the coefficient, A the area in square feet, and h the difference in water levels on both sides, varies from about 0.70 to 0.80, depending on the amount of contraction and the value of h. A difference in water level of about 3 feet will produce a velocity through the gates of about 10 feet; at times a much greater velocity can usually be obtained, but a high velocity through the gate openings does not give a correspondingly high velocity through the sluiceway channel unless the area of sluicegate opening is not much smaller than the cross-sectional area of the sluiceway. The scouring velocity through the sluiceway channel, developed by opening the sluicegates, must be at least greater than the maximum velocities obtained in the sluiceway channel during the depositing period,

when the sluicegates are shut and the canal headgates fully opened. A scouring velocity of usually not less than 5 to 10 feet per second, depending on the character of the material and the length of time the sluices can be left open, will usually be necessary; to obtain this velocity, the proportion of gate area to sluiceway area which must be provided will depend on the obtainable velocity through the sluicegates, which is dependent on the difference in water levels; for instance, if a velocity of 20 feet through the sluicegates is obtainable, the area of gate opening must be $\frac{1}{2}$ of the sluiceway cross-sectional water area to obtain a scouring velocity of 10 feet per second.

The following safe rules may be deduced for the determination of the capacity of the scouring sluices:

First.—Make the capacity at least equal to the normal stream flow during the irrigation season and preferably equal to the minor normal flood flow.

Second.—Make the cross-sectional water area of the upstream sluiceway channel or basin at least equal to and preferably 25 or 50 per cent. larger than the cross-sectional water area of the canal, for all stages of the river water level.

Third.—Provide sluicegate openings of sufficient area to give a scouring velocity in the sluiceway channel at least greater than the maximum velocity, obtained in the channel during the depositing period, when the sluicegates are closed and canal headgates opened to deliver full supply in the canal. A scouring velocity of 5 to 10 feet per second will generally be necessary.

Sluiceway Channels.—The main purposes of the upstream sluiceway channel, as indicated above, are:

First.—To form a basin in front of the headgates, in which the water will move at a decreased velocity, when the sluicegates are shut and headgates open.

Second.—To concentrate and confine the scouring effect of the high velocity developed through the sluicegates.

The downstream sluiceway channel confines the erosive effect of the water discharge through the sluicegates.

The floor of the sluiceway channel must be kept as low as feasible with respect to the sill of the canal headgates to prevent the entrance of the coarse sediment in the canal. It is usually placed either level with the weir floor, but at least 4 feet below the canal headgate sill; if necessary it may be placed lower than the weir floor.

The width of the sluiceway between the headgates and division wall is determined from the required capacity of the scouring sluices in accordance with the rules and principles stated above. The length of the upstream sluiceway channel measured from the sluicegates must be sufficient to place the entrance to the channel well above the canal headgates. To obtain this it will usually be sufficient to extend the upstream end of the division wall to a point at a distance above the upstream edge of the canal headgate opening equal to the total width of the headgates.

The division wall may be a rock fill or a concrete wall, with its crest at least as high as the weir crest and preferably as high as maximum flood water level, for it will then separate the basin from the remainder of the stream at all river stages, which is of special advantage during flood flows when the greatest amount of sediment is carried.

The upstream floor may be carried to the end of the division wall, or only to the upstream edge of the canal headgates, in which case the channel above will be completed with a riprap floor between the division wall and the stream bank or upstream wing of the canal headgate. The downstream floor must extend a sufficient distance to protect the stream bed from strong erosive currents, which have a tendency to cut a hole at the downstream end of the floor. W. G. Bligh recommends that the length of the downstream floor be 50 per cent. longer than that of the weir floor. The downstream division wall will usually extend to the end of the floor, but where the stream bed is rock or stable material, the downstream floor may be stopped at the weir floor and the division wall omitted. The base of the division wall must extend down to a firm foundation or be built on a foundation closed in with piles.

The design of the thickness of the upstream and downstream floors, of the cut-off walls, and of the riprap river bed protection must follow the same principles as used in the design of the diversion weir, except for the increase in the length of the downstream floor, as stated above. The upstream floor is thinner than the downstream floor, because the hydrostatic uplift pressure on it is more than balanced by the downward pressure of the weight of the water.

To complete the sluiceway channel, it is either necessary to protect the stream bank with a retaining wall or slope protection or both, parallel with the division wall and extending upstream

and downstream beyond the end of the division wall. The slope protection may be a reinforced concrete lining, a riprap lining or a brush lining weighted down with rocks or concrete blocks tied together with wire cables.

Piers, Gates, and Operating Platform.—The sluiceway channel is usually divided by piers into bays or openings regulated by gates; the openings must be unobstructed up to the maximum flood flow water level. On a number of projects in India, collapsible gates or shutters have been used, in order to avoid the use of intermediate piers, but it has been found difficult to make them work successfully for heights of gates exceeding 6 to 8 feet. In India the modern tendency is to use large openings from 10 to 20 feet or more in width. The piers are usually of the gravity type; they are carried down below the sluiceway floor to a solid foundation, or to a foundation enclosed in sheet piling, and extend above flood water level. The piers have usually a trapezoidal profile, the thickness, base width and top width being proportioned to produce a resultant pressure falling inside of the middle third of the base. W. G. Bligh recommends an empirical rule for the thickness of piers which conforms with the practice in India; this rule gives a thickness of pier ranging from .21 to .31 of the width of the opening; an average value of .25 may be used.

The piers are usually provided with two sets of grooves; the upstream set of grooves is placed near the nose of the pier and is used for the insertion of emergency gates or of flashboards; the lower set of grooves is placed a short distance back of the flashboard grooves and is used for the regular lift gates.

On top of the piers is usually supported a permanent or removable operating platform and the necessary framework or overhead structure required to permit raising the gates to their full height above the flood water level. In some cases, as illustrated further by the sluicegates used for the Granite Reef Diversion Weir in Arizona, a special lifting mechanism is used, which does away with the necessity of the operating platform and overhead structure. The gates must be of the undershot type and not of the overpour type. Gates of wide span are considered desirable for sluicegates. Special types of gates designed to be operated by a comparatively small lifting force are preferable. The Stoney patent gate with roller bearings and the Taintor type of gate are specially well adapted. The Stoney patent gate is

illustrated by the sluicegates of the Murrumbidgee Weir in Australia, and of the Yuma project diversion weir on the Colorado River; these examples are described further. The Taintor type of gate and the usual type of gate-lifting devices are described in the discussion of canal headgates. Special types of gate-lifting devices are illustrated by the examples of sluicegates presented below.

Examples.—In the preceding discussion special emphasis has been laid on open scouring sluices as used on streams carrying much silt; these conditions exist in the United States, on such streams as the Salt River in Arizona, the Colorado River in California and Arizona. For streams carrying little silt the necessity for scouring sluices is not so great and the practice regarding their design varies widely and follows no well-defined principles. In many cases they consist of undersluices formed of comparatively small openings through the dam; for instance, the undersluices at each end of the Capay diversion weir on Cache Creek consist of two openings, each 4 feet wide and 4 feet high, with the sill 8 feet below the permanent crest of the weir and 5 feet below the sill of the headgates. The undersluices at each end of the diversion weir for the North Platte project consists of two openings, each 5 feet 9 inches wide, 6 feet high, with their sill 14 feet lower than the crest of the weir and 7 feet lower than the sill of the canal gates. In these cases no division wall is built to form a sluiceway channel in which the scouring effect may be concentrated.

The following descriptions present examples where the large amount of sediment carried by the river water required efficient sluices, and examples which contain features which may be adapted to special conditions.

Scouring Sluices of Granite Reef Diversion Works, on Salt River, Arizona.—The Granite Reef diversion weir, which has been previously described, diverts water from the Salt River to supply two canals, one on the south side of the river with a capacity of 1,200 cubic feet per second, the other on the north side with a capacity of 2,000 cubic feet per second. The Salt River has a very variable flow and carries a large percentage of sediment, especially during flood flow. The accompanying drawing (Fig. 33) shows a plan of the headworks for the south-side canal.

The canal has a full supply capacity of about 1,200 second-

feet The sluiceway channel, about 40 feet wide, is formed between the division wall and the face of the headgate wall, which is extended as a retaining wall upstream and downstream to join with the warped wings and riprap protection of the river bank. The crest of the division wall is level with the crest

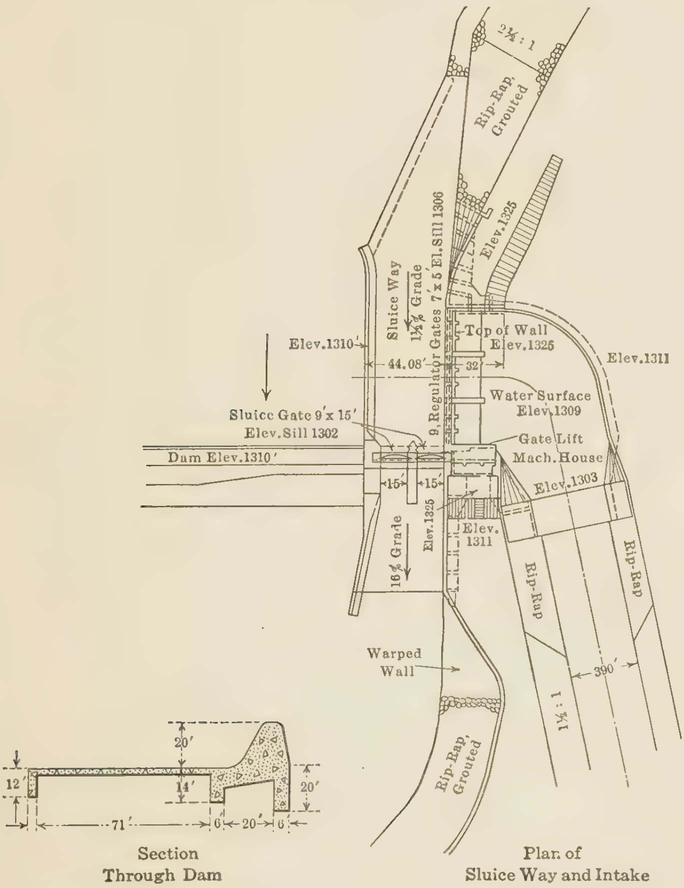


FIG. 33.—Scouring sluices at Granite Reef dam. Salt River Project, Ariz.

of the dam. The upstream floor of the sluiceway slopes toward the sluicegates on a grade of 1½ per cent., and at this point is 8 feet below the crest of the dam and 4 feet below the sill of the canal headgates. The canal headgate wall is divided into three main spans by two piers 3 feet thick, spaced 26 feet on centers.

The wall is open at the bottom up to the height of the gate openings, 5 feet from the sill, and each span opening is subdivided by two intermediate beams, 1 foot thick, spaced 8 feet apart, which form the guides for the gates. This forms nine gate openings, 7 feet wide and 5 feet high. The sluiceway gates are in line with the axis of the dam; there are two gates, each 15 feet wide and 9 feet high.

The headworks for the north side canal are of the same type

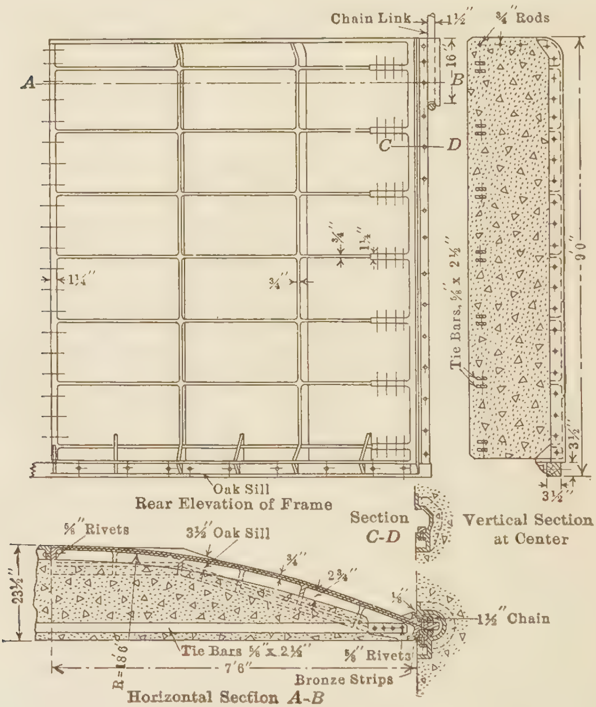


FIG. 34.—Details of sluice gate, Granite Reef Diversion Works. (*Eng. News*, Jan. 1, 1909.)

and design, varying only in the capacity and minor details. This canal has a full supply capacity of 2,000 second-feet. The sluiceway channel is about 80 feet wide, and is closed with four sluicegates of the same dimensions as on the south side. The canal intake is regulated with 18 gates.

The cross-sectional area of the sluiceway channels from the floor to the crest of the weir and division wall is about 320 square feet on the south side and 640 square feet on the north

side; these are about equal to the cross-sectional area of the respective canals.

The cross-sectional area of the sluicegate openings, with the sluicegates fully opened to a height of 8 feet, is 240 square feet on the south side and 480 square feet on the north side, which gives ample capacity to obtain the required scouring velocity in the sluiceway channels.

The sluicegates are scoured out once a week, and it has been

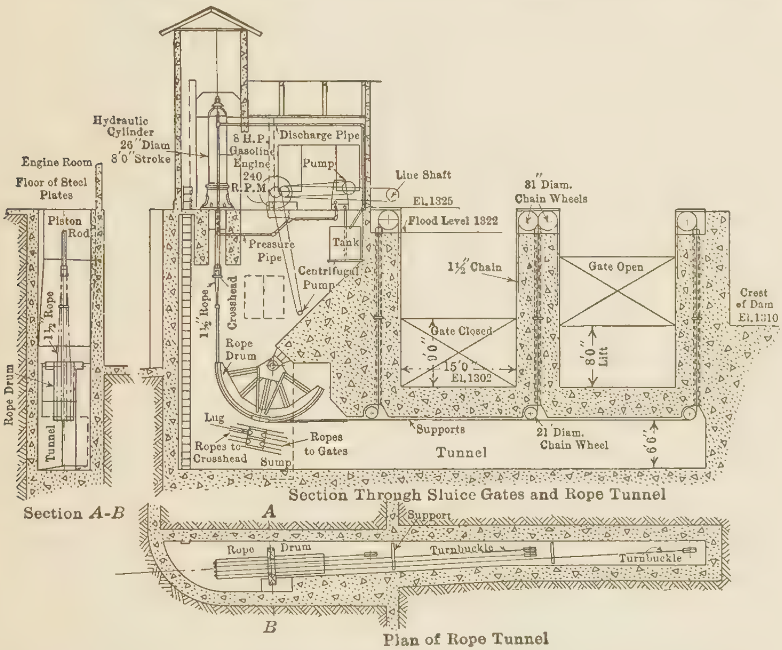


FIG. 35.—Details of operating mechanism of the sluice gates. Granite Reef diversion dam, Ariz. (*Eng. News*, Jan. 1, 1909.)

found that in this time the sluiceway is well filled with sand deposits.

The sluicegates and operating mechanism are of an unusual design. Each sluicegate (Fig. 34) is made of a ribbed cast-iron curved shell, ¾ inch thick, in two halves riveted together. The water face is a smooth convex surface; the downstream face is ribbed, and on this side, tie-bars, ⅝ × 2½ inches, placed in sets of two, opposite each horizontal rib, connect together the edges of the curved shell. The ribbed face and tie-bars are encased in concrete, which gives a total weight to each gate of

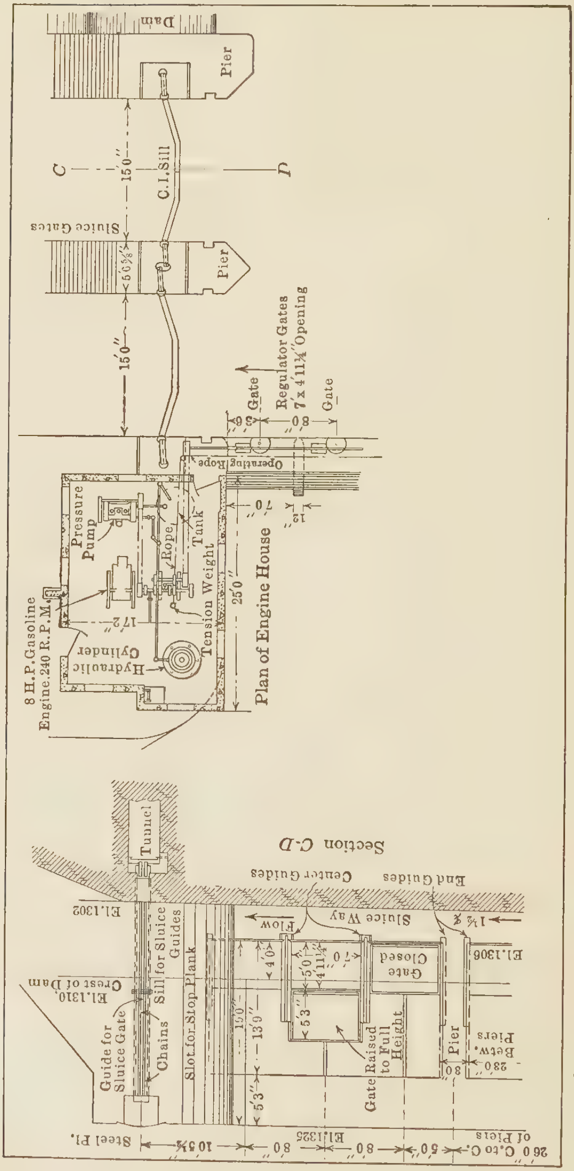


FIG. 36.—Engine house, sluice gates and canal headgates. Granite Reef diversion dam, Ariz. (*Eng. News*, Jan. 1, 1909.)

about 30,000 pounds. This weight is necessary for the gates to close by their own weight. Each gate is hung to two $1\frac{1}{2}$-inch chains, which pass over 31-inch chain sheave wheels, supported

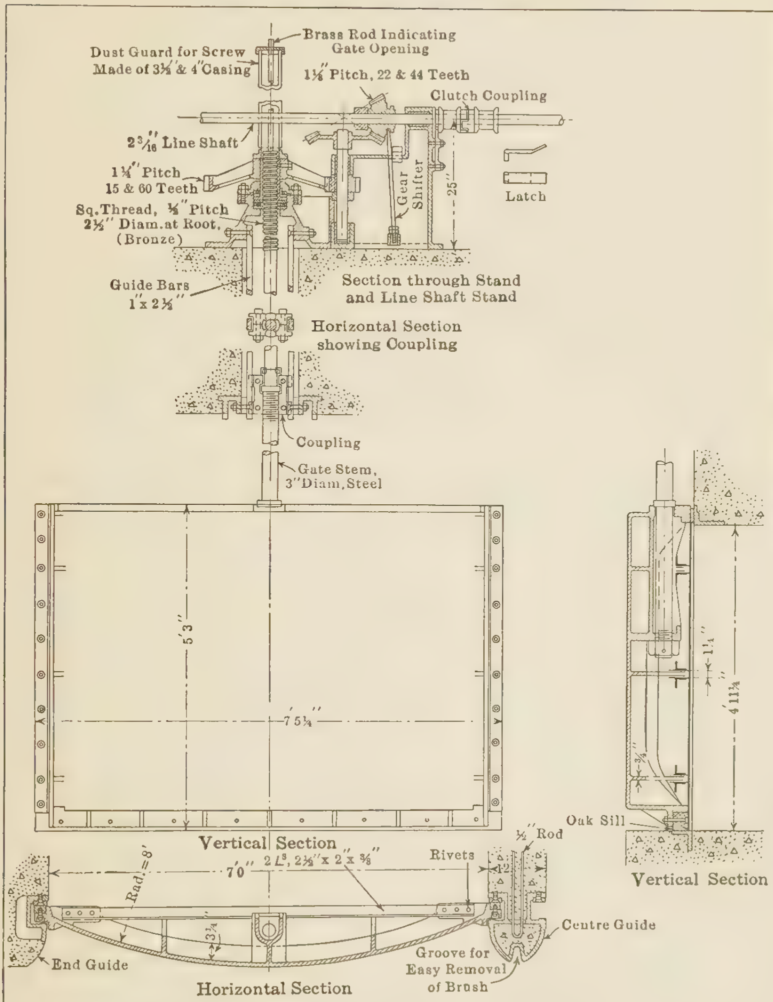


FIG. 37.—Details of regulator gates. Granite Reef dam, Ariz. (*Eng. News*, Jan. 1, 1909.)

at the top of the piers, and extend down through a groove in each pier to a rope tunnel under the sluiceway floor; then the chains of each pier, in sets of two, pass around a 21-inch chain

wheel and each chain connects with a $1\frac{1}{2}$-inch steel rope; then the steel ropes are all connected to the periphery of a steel drum 7 feet in diameter and extend around the drum to connect with the lower end of the piston rod of a hydraulic cylinder, 28 inches in outside diameter, capable of producing a cylinder pressure of 700 pounds per square inch for the operation of the four sluicegates on the north side (Figs. 35 and 36). The cylinder is operated by a pressure pump and an 8-horsepower gasoline engine. The engine also operates the canal headgates by transferring its power to a horizontal shaft, which passes near each gate stand and transmits the lifting or closing power to the gate stem through a system of bevel gearing. The canal headgates are formed of curved cast-iron shells, smooth on the convex or water side, with three sets of angle tie-bars on the concave side (Fig. 37).

Scouring Sluices of Yuma Project Diversion Works, on Colorado River, Arizona-California.—As previously stated, these diversion works raise the water level of the Colorado River to divert water into two canals, one on each side of the river. The area to be irrigated is partly in Arizona and partly in California, aggregating about 131,000 acres. The largest part of the area to be irrigated is on the Arizona side of the river and south of the Gila River. As originally planned, the canal supplying this area was to head at the east end or Arizona end of the weir and be carried under the Gila River, and a smaller canal heading at the west end or California end of the weir was to supply the smaller area in California. The difficulties of crossing the Gila River were objectionable, and a more favorable and economical solution was obtained by reversing the position of the two headings. The larger canal heading was placed on the California side; the main canal serves the irrigable area in California and is carried under the Colorado River at a point a short distance below the junction with the Gila River by a large inverted siphon to supply all the land on the Arizona side south of the Gila River. The smaller canal heading was placed on the Arizona end of the weir and the canal leading away from it commands the smaller area north of the Gila River. Other than this reversal of the position of the headings, only small changes were made in the original plans, and the descriptions of these works as given in the articles included in the list of references are based on the original plans.

On account of the large amount of silt carried by the river

water, the principal consideration was to so design the sluiceway and intake to each canal that the least amount of silt possible be carried into this canal. To obtain this result, each heading consists of a sluiceway channel, excavated in the solid rock bank, around the end of the weir, regulated at the downstream end with wide Stoney roller gates, and of an overflow skimming wall, formed as part of the uphill side wall of the sluiceway channel, divided by piers into a number of bays, regulated by means of horizontal flashboards. The floor of the sluiceway channel is lined with 6 inches of concrete and the sides are concrete retaining

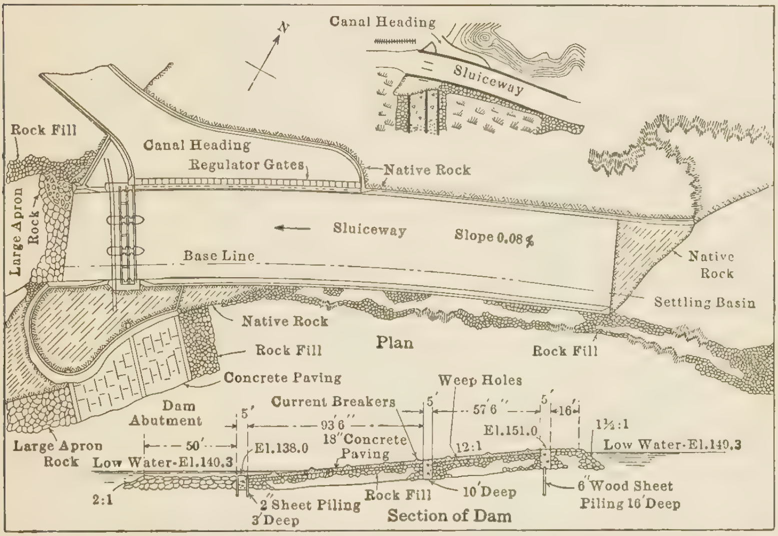


Fig. 38.—Scouring sluices and headgates at Yuma Project diversion works on California side of Colorado River. California–Arizona.

walls. On the Arizona side the channel is 18 feet deep, 40 feet wide at the bottom and 52 feet wide at the top; it is contracted at the downstream end to a width of 33 feet 4 inches, closed and regulated by a single Stoney gate. The channel extends for about 570 feet upstream from the sluicegate. The canal headgates formed in the uphill side wall of the sluiceway, upstream from the sluicegate, consist of eight bays, each 7 feet 6 inches wide, separated by piers 1 foot thick. The sill of the bays is 10 feet above the sluiceway floor and 3 feet lower than the crest of the weir; the horizontal flashboards may be inserted between the piers to raise the sill and skim the top water up to the full height

of the sluiceway channel. The details of the Stoney sluicegate and canal headgates and the method of operation are the same as for the California heading, which is described more fully.

The sluiceway channel on the California side (Fig. 38) is 18 feet deep, 116 feet wide at the bottom, and 128 feet wide at the top, contracting at the downstream end to a rectangular cross section 116 feet wide, divided by piers 8 feet wide into three sluicegate openings, each 33 feet 4 inches wide, and regulated by a Stoney rollergate. The sluiceway channel extends for about 650 feet upstream from the sluicegates. The canal headgates are formed as on the Arizona side and consist of 35 bays, with the permanent sill 9 feet above the sluiceway floor.

The method of operation consists in keeping the sluicegates shut when taking water into the canal, during which period the slow velocity encourages the deposit of sediment in the sluiceway channel—then opening the sluicegates to scour out the deposited material. The sluicegates may also be used to regulate the flow of water over the canal gates. Assuming that the water level of the river is raised up to the weir crest and that the flashboards of the canal gates be adjusted to skim the top water for the full capacity of the canal of about 1,400 second-feet; then the depth of water in the sluiceway will be equal to the height of the weir crest above the sluiceway floor, or 13 feet; this gives a cross-sectional water area in the sluiceway channel, when not diminished by silt deposit, of about 1,560 square feet, or a velocity of about 0.9 feet per second, which is sufficiently low to favor the deposition of the coarser sediment. E. D. Vincent, the resident engineer, states that during the completion of the weir across the river, in the spring of 1909, the full capacity of the sluiceways was utilized; this combined capacity was about 18,000 second-feet. The full capacity of the California sluiceway alone is probably about 15,000 second-feet, which gives a scouring velocity of nearly 10 feet per second.

The sluiceway gates and their operating mechanism are of special interest as illustrating a type of gate suitable for very large openings, and for this reason a rather complete description is given. The three sluicegate openings on the California side are formed between two intermediate piers and the abutment walls; in which 8 feet upstream from the grooves for the main service gates, are sets of grooves for an emergency gate, which may be moved through slots in the piers to any one of the

gate openings (Fig. 39). The operating mechanism for this gate is carried on a travelling car, which is supported on a girder track (Plate VI, Fig. A). The lifting mechanism for the main service gates is supported on a floor level with the top of the piers. Each sluicegate (Fig. 40) is 34 feet 9½ inches wide, and 17 feet 11¼ inches high; the upstream face is made of metal plates riveted to four horizontal girders and five vertical girders. The horizontal girders transmit the water pressure to the side edges of the gate; one of these girders is placed along the bottom edge of the gate, another 5 feet from the bottom, another 10 feet from the bottom, and another along the top edge. Two of the vertical girders are placed along the side edges of the

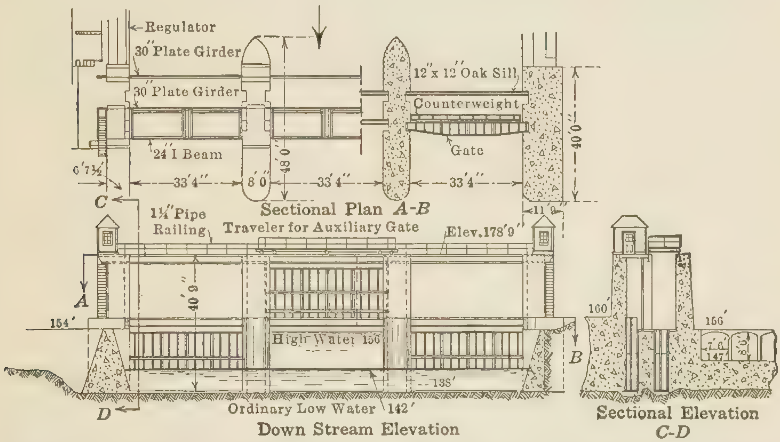


FIG. 39.—Details of sluice gates at Laguna diversion works, Yuma Project. Ariz.-Calif.

gates, one in the center and the other two in between. The girders are built of a web plate with flange angles and angle stiffeners. The grooves for the gates are built of heavy iron castings, forming a channel 16 inches deep and 37¾ inches wide, with ribs on the outside, imbedded in the concrete. On the downstream side of the gate and riveted to the ends of the horizontal girders are bearing strips with a raised finished bearing surface, 6½ inches wide. Between this bearing surface and a similar raised surface of about the same width, formed on the downstream face of the groove casting, is a train of rollers, through which the water pressure is transmitted (Fig. 41). Each roller train consists of 26 cast-iron rollers, 6 inches in diameter,

$5\frac{11}{16}$ inches wide, with bronze bushings revolving on $1\frac{1}{4}$-inch steel pins, fitted between two $\frac{3}{8} \times 6$-inch wrought iron

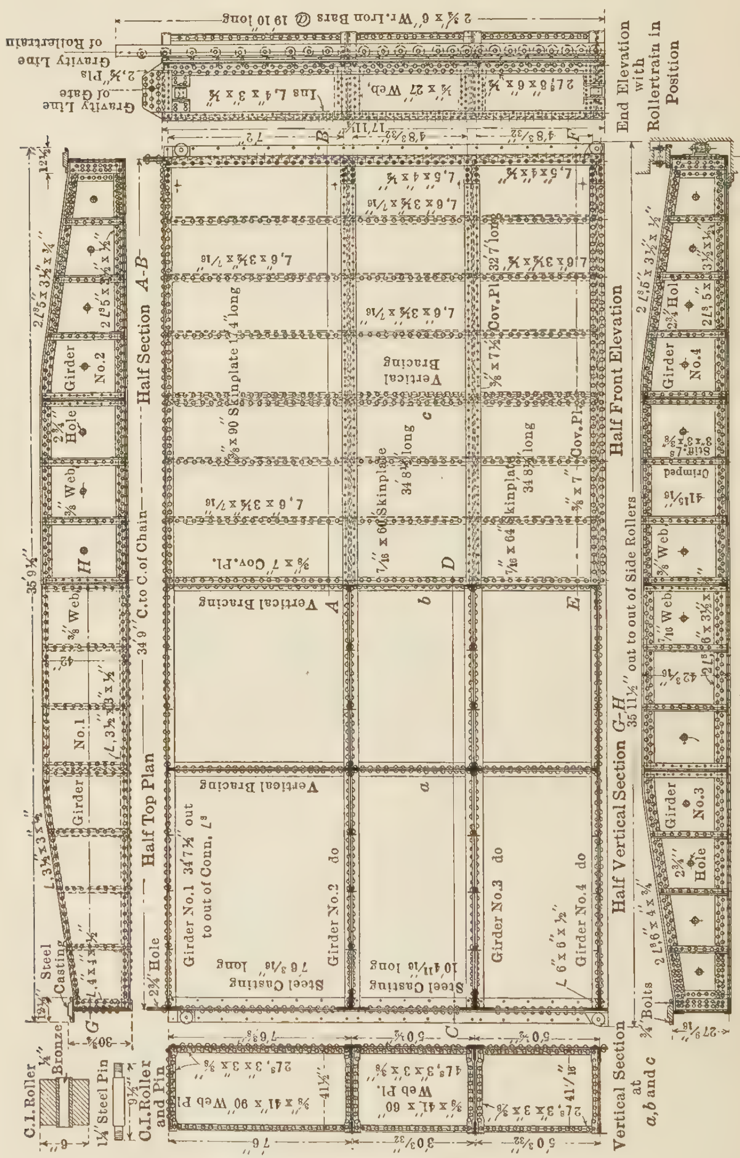


FIG. 40.—Body of sluice gate and roller train. Laguna dam, California end. (P. 218, *Eng. News*, Feb. 27, 1908.)

plates, 19 feet 10 inches long. To prevent side motion or binding of the gate, rolling contact is obtained between the side

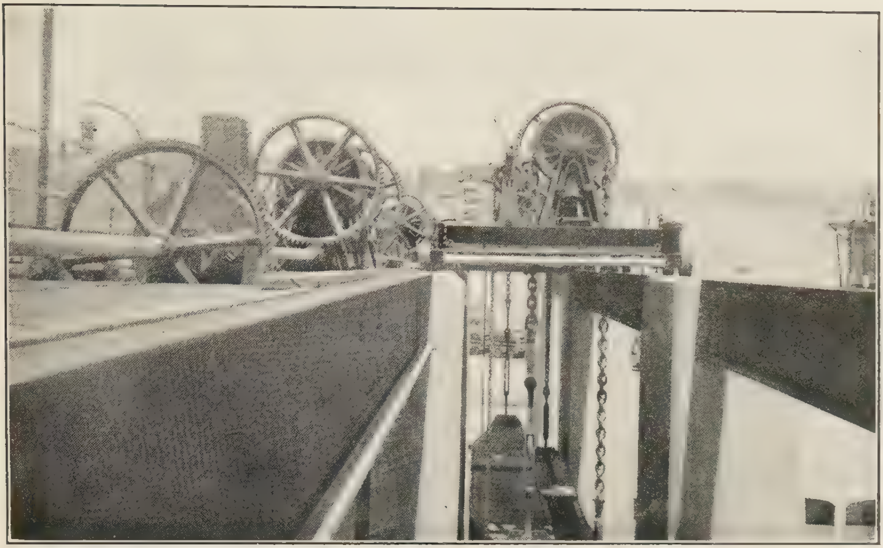


FIG. A.—Lifting mechanism for sluice gates of Laguna Diversion Works California end of Yuma Project, Ariz.

FIG. B.—Scouring sluices and Murrumbidgee Diversion Works, New South Wales, Australia.

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edges of the gate and a raised surface 6 inches wide, formed on the back face of the groove, by four cast-iron rollers, 6 inches wide and 6 inches in diameter, placed at the top and bottom of the

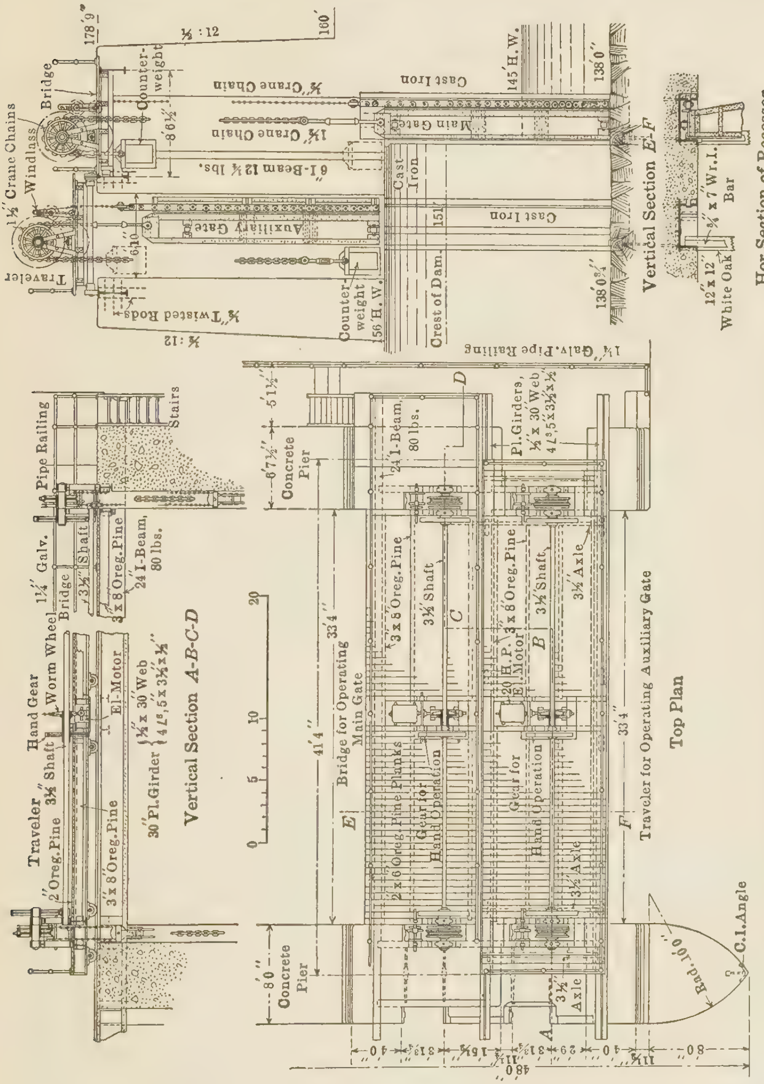

FIG. 41.—Sluice gate, with operating machinery. Laguna dam, California end. (P. 218, *Eng., News* Feb. 27, 1908.)

edges of the gate. To form close contact with the side edges of the gates and the piers or abutment wall, strips of 3 × 3-inch Oregon fir are secured to the gate along the upstream edges of

the gate and against the flange of the casting. The lower edge of the gate is reinforced with the leg of an angle and a narrow cover-plate, and is finished to give a tight seat on the metal plate, which covers the wooden sill imbedded and anchored to the rock.

The gate is suspended at each of the two top corners by 1½-inch crane chain (Fig. 41); the two chains of each gate pass over sheaves, keyed to and supported on bearing shafts, and are connected at the other end to a counterweight made of plates and angles, 27 inches square, 34 feet 9 inches long, loaded with cast-iron weight to give a total weight of about 48,500 pounds, equal to the weight of the gate and a pair of roller trains. The upper end of the roller train contains a pulley, which is suspended on a ½-inch chain; one end of the chain passes over a sheave pinned to the same bearing shaft as the corresponding gate chain and secured to the same counterweight; the other passes over a cast-iron drum on the operating platform (Plate VI, Fig. A).

There are two sets of sheaves supported on the top of each pier. On the axle or bearing shaft of each set is mounted a main spur gear wheel, which meshes with a bronze pinion, fixed to the operating shaft, and through which the operating force is transmitted (Fig. 41). The operating shaft extends between the two sets of sheaves of each gate, and has fixed to it on its center a worm wheel which engages a worm shaft, on which the operating force is applied by connection with the shaft of a 20-horsepower motor. Provision is also made for operation by hand, through a pinion acting on a main spur gear wheel fixed to the shaft. The travelling car used for the emergency movable gate carries a similar operating device.

Scouring Sluices of Murrumbidgee Diversion Works, New South Wales, Australia (Fig. 42 and Plate VI, Fig. B).—The collapsible diversion weir, consisting of Chanoine shutters, has been previously described. The scouring sluices consist of two sluiceway channels, formed at the end of the weir adjacent to the canal headgates, each giving a clear width of 40 feet, regulated with a Stoney roller gate of the same width. The sluiceway channel farthest from the canal headgates is provided at the downstream end with lockgates to form a lock chamber. The main purpose of the sluices, in addition to providing for river navigation, is to permit the regulation of the stream flow and dis-

posal of the surplus water during minor freshets, when the Chanoine shutters are raised. These are the determining factors which account for the large area of sluiceway channel. The sluicegates are of the Stoney type, each 40 feet wide and 16 feet high. The gates are built of ½-inch steel plates, riveted to

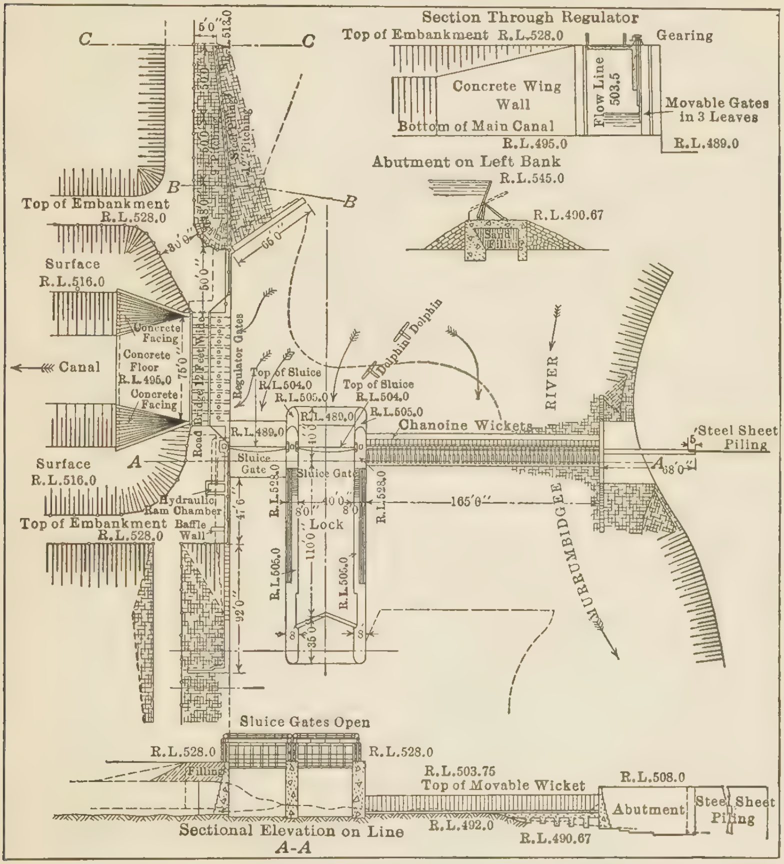

FIG. 42.—Murrumbidgee diversion works. New South Wales, Australia.

and stiffened by five inverted bow-string girders, all of the same dimensions but spaced closer together toward the bottom of the gates. The gates work between cast-iron grooves, built in the concrete piers, extended by structural steel columns, to a total height of 55 feet above the sill, to support the two steel girders, which carry the overhead bridge. The gates may be

lifted to give a maximum clear height of opening of 39 feet. Each gate weighs about 20 tons; they are hung on sprocket chains passing over sheaves on the operating platform, and are counterbalanced by cast-iron weights, suspended to the other end of the chains, which travel between channel grooves fixed to the overhead framed columns and into a well provided in each pier. The pressure on the gates is carried to sets of travelling rollers placed in the grooves, between the edge of the gates and one edge of the groove. Each train of rollers is suspended by ropes passing around movable pulleys attached to its upper end. One end of the pulley rope is secured to the gate and the other to a bar, supported freely on a bracket 28 feet above the gate. By this means the train of rollers is raised with the gate, but travels only half the distance. When the gate has been raised 28 feet and the train of rollers 14 feet, a bracket on the gate engages the bar to which the other end of the pulley rope is attached, and the train of rollers then raises as fast as the gate. The gates when closed rest on a sill bedded in the concrete floor; the sides of the gates are provided with steel staunching bars, suspended so that the water pressure renders the gates practically water-tight. The gates are raised and lowered by a bevelled wheel gearing, with roller bearing, operating on the overhead shaft which works the sprocket chain at each end of the gates. They can be operated by two men and require about the same force to raise as to lower them.

Scouring Sluices of Corbett Diversion Works, Shoshone River, Wyoming (Fig. 43).—The diversion weir, previously described, is about 15 miles downstream from the Shoshone dam, which forms a storage reservoir of 456,000 acre-feet. The Corbett weir diverts the water from the river into the head of the Corbett tunnel, which has a capacity of about 1,000 second-feet. The tunnel is $3\frac{1}{3}$ miles long and carries the water to the head of the Garland main canal. The entrance to the Corbett tunnel forms two gate openings, each 5 feet wide and 10 feet high, and closed with a main gate with provision for emergency flashboards fitting in grooves above the main gate. A skimming wall, forming a basin or pool in front of the gates, is used to divert into the canal the surface or top water of the river. A sluiceway channel is formed in front of the skimming wall by a reinforced concrete division wall which extends above the high water level. The flow through the channel is regulated by three

sluicegates, which control the three openings through the lower part of the closing wall across the downstream end of the sluiceway.

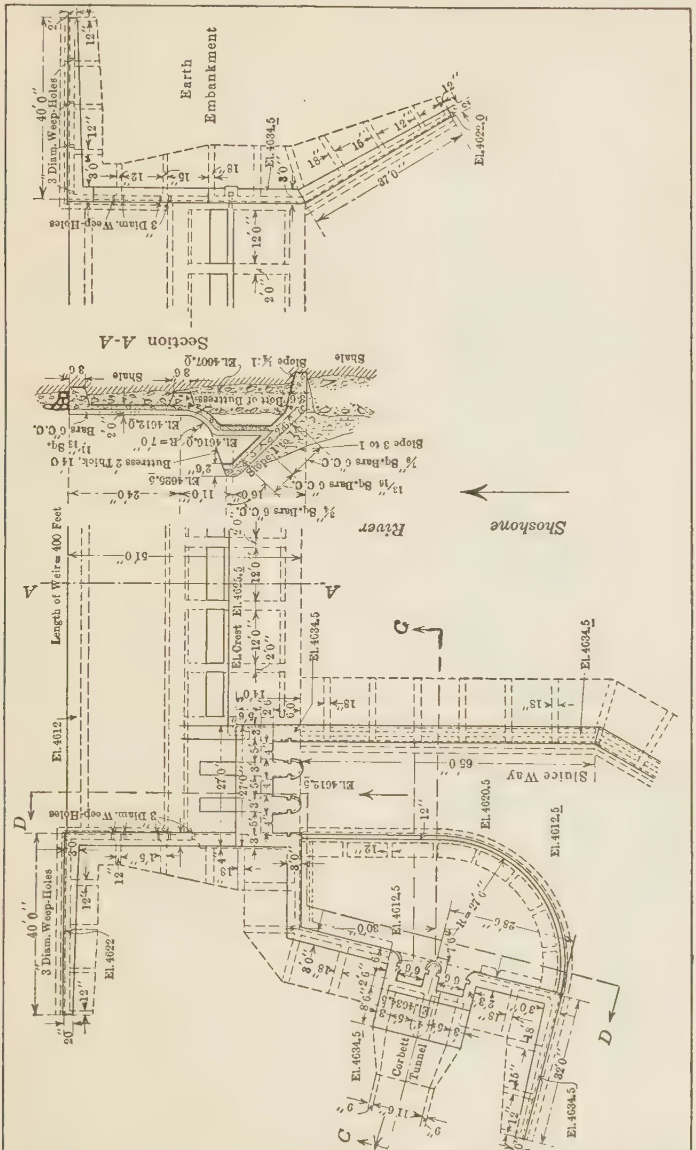

FIG. 43 A.—Plan of Corbett dam and headworks at Corbett tunnel. Shoshone Project, Wyo.

The sill of the sluiceway opening is 6 inches above the level of the weir floor, and 8 feet lower than the crest of the skimming wall.

A study of the obtainable depositing and scouring velocities produced by this arrangement of skimming wall and sluicegates

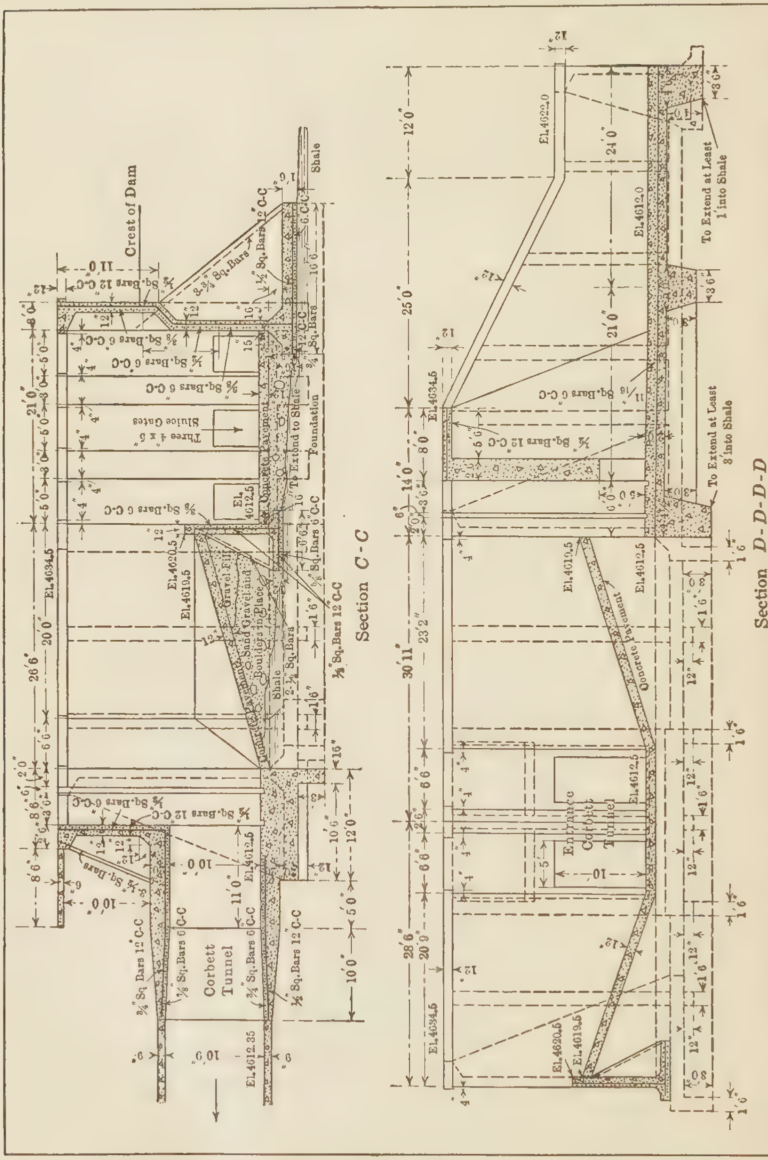

FIG. 43 B.—Longitudinal and transverse sections of sluiceway of Corbett Tunnel diversion works. Shoshone Project, Wyo.

is interesting to illustrate the action of scouring sluices. With the sluicegates and the tunnel gates closed, the sluiceway channel

and the basin are separated from the main part of the stream and will form a practically still body of water, in which very little of the sediment carried by flood flows will enter. The greatest velocity obtained in the sluiceway channel, with the scouring gates shut, is when the maximum full supply is being taken in the canal with the water level of the river at the minimum elevation required to force the water over the skimming wall and through the tunnel gates. Assuming a coefficient of discharge through the gates of 0.8, the water level in the basin, to deliver a full supply of 1,000 second-feet through the gates, must be about 2.5 feet above the top of the gate opening or at a level of 4.5 feet above the crest of the skimming wall. The flow of water over the skimming wall is then that of a submerged weir, and for a crest length of about 80 feet requires a depth of water on the crest of the skimming wall of very nearly 5 feet; this brings the required water level in the sluiceway channel at the same level as the weir crest. For these conditions and assuming that the full water cross-sectional area of the sluiceway is equal to its width of 21 feet multiplied by the depth from the water level to the sluicegate sill, 13 feet, then the maximum velocity obtained in the sluiceway when diverting 1,000 second-feet in the tunnel is about 3.6 feet per second. This velocity is greater than is desirable; it will carry in the sluiceway coarse material which, when deposited, will require a much larger velocity to be scoured out. The full scouring discharge obtained through the sluicegate openings when the upstream water level of the river is level with the weir crest (assuming the most favorable conditions such as a coefficient of discharge of 0.8 and no submergence), is about 1,270 second-feet, which, although producing a velocity of about 20 feet per second through the gate openings, results in a scouring velocity through the full cross-sectional area of the channel of about 4.5 feet per second or only 25 per cent. larger than the depositing velocity. The filling of the sluiceway channel by the deposition of sediment will, however, gradually decrease its water cross-sectional area, which will result in a greater scouring velocity when the sluicegates are first opened, and as the channel is gradually enlarged by the scouring out of the deposited material, the velocity in the channel will gradually diminish.

It is interesting to notice that a lowering of the water level of the river on the upstream side will, within certain limits, in-

crease the scouring velocity in the sluiceway channel. For instance, with the water surface level with the crest of the skimming wall, the discharge through the sluiceway channel with the gates fully opened and no submergence will be nearly 900 second-feet, which will produce a scouring velocity in the sluiceway channel for the full water cross-sectional area, 21 feet wide and 8 feet deep, of about 5.4 feet per second.

The Shoshone River water carries little sediment; the measurements taken by the U. S. Geological Survey from April 2, 1905 to March 30, 1906 give for the maximum amount of sediment carried in suspension 258 parts per million by weight and usually considerably less. These results were obtained before the large storage reservoir was formed by the construction of the Shoshone dam. The effect of this storage is to remove a large part of the sediment; there can be therefore little necessity for the scouring sluices. On the other hand, if the amount of sediment were as large as on some of the streams of Arizona or New Mexico, the scouring sluices should be designed to give a larger scouring velocity in the sluiceway channel The efficiency of the sluices would also be increased by placing gates on the crest of the skimming wall, because the scouring velocity developed by opening the sluicegates would then be concentrated to the body of water in the sluiceway channel formed in front of the gates by the division wall, while without these gates the body of water spreads over the skimming wall. This may have been the reason for changing the original plans of the Granite Reef headworks, which at first contemplated a skimming wall formed on the present axis of the canal headgates, later changed to the canal headgates located a short distance downstream from the skimming wall on a line with the axis of the dam. The type of overpour flashboard gates, such as used for the canal headgates on the Yuma project, is the most efficient type to keep the sediment from entering the canal.

FISH LADDERS

The laws of many states require that fish ladders be placed in all weirs or dams to permit the fish to pass upstream. For rivers whose summer flow is small the water may be confined to a channel upstream and downstream from the sluiceway. For these cases it is necessary to place the fishway adjacent to the

sluiceway. A fish ladder consists of a series of basins forming a set of steps. These basins are connected by means of openings

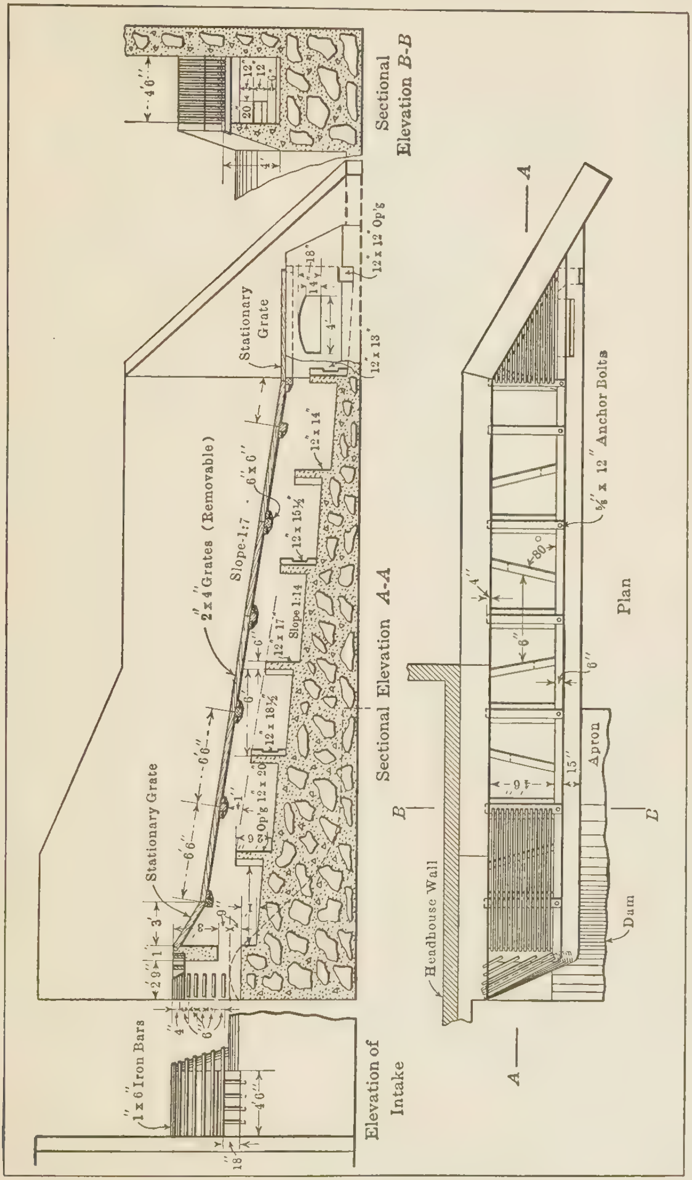

FIG. 44.—Detail of fish ladder at diversion dam of Yakima-Sunnyside Project, Wash.

so that the fish can pass from the lower basin to the upper basin. The inlet to the lower basin must lie lower than the low water

level on the downstream side of the weir and the outlet to the upper basin is on the upstream side of the weir below the weir crest. To insure a full supply in each basin the upstream opening for each basin is made a little larger than the downstream

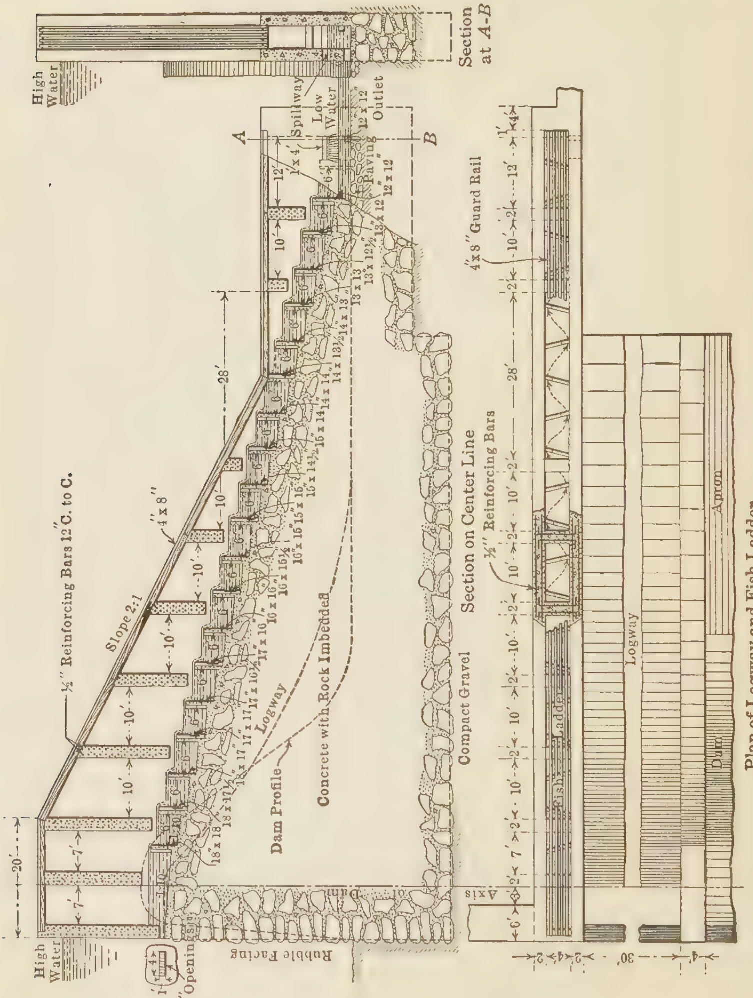

FIG. 45.—Detail of fish ladder at diversion dam of Boise Project, Idaho.

opening, so that the openings gradually diminish from a maximum at the top to a minimum at the bottom. The openings in each basin are usually made at the bottom and at opposite corners so that the fish must follow a zigzag path. This arrangement leaves

one corner where the water is comparatively still where the fish can rest. This is necessary, especially for high weirs. The difference in elevation between basins regulates the velocity through the openings and should not be in excess of about 18 inches. The slope of the fishway should not be steeper than 1 foot vertical to 4 horizontal. The width of the fishway is from 4 to 5 feet. The baffles forming the basin are often placed on a 60 to 80 degree angle to the longitudinal axis so as to form trapezoidal basins. The average length of these basins is about 6 feet and the depth of water from 2 to 4 feet. The minimum sized opening between basins should be 12 by 12 inches. The fishway should be covered with a grating or screen to prevent interference, but the light must not be shut out or the fish will not use it. The design of fishways is illustrated by the fish ladder in the diversion dam of the Sunnyside project, Washington (Fig. 44) and the fish ladder in the Boise project diversion dam (Fig. 45); (see also Plate I, Fig. A). Crude types of fish ladders, consisting simply of an inclined shallow wooden flume, divided into basins by baffles, are shown in Plate II, Fig. D and Plate III, Fig. D.

LOGWAYS

In some states where logging is practised laws have been enacted making it necessary to provide for free navigation of streams for logging purposes. This requires ready means for passage of logs either over or through the dam. When overflow weirs are on streams carrying enough water to insure a large depth of water over the crest at all times during the year, it may not be necessary to provide a logway, as the logs will pass directly over the main body of the weir. A logway is generally formed by lowering the crest of the dam for a short section adjacent to the fish ladder and scouring sluice. The sides of the logway channel are formed by two walls running parallel with the direction of flow, one of which may be the side of the fishway or the abutment to the dam. In many cases to avoid an excessive waste of water, it is desirable to build the logway as narrow as possible and the inlet is controlled by a gate which may be opened only when necessary. The width of the channel may be as small as 6 to 8 feet and usually ranges from that up to 20 or 30 feet, depending on the amount and character of logging. The sill of the logway is made at least 4 feet lower than the crest of the weir. To guide

the logs into the logway a timber boom starts from the head of the logway on a small angle with the direction of flow and extends upstream a distance sufficient to intercept the floating timber and guide it to the logway. The design of the logway is indicated in the above plan of the fish ladder in the Payette-Boise diversion weir (Fig. 45) and is also shown in Plate I, Fig. A.

REFERENCES FOR CHAPTER II

Report on the Deposit and Scour of Silt in the Main Line, Sirhind Canal, and on the Silt Experiments from 1893–1898—Punjab Irrigation Branch Papers, No. 9.

The Irrigation Conference, Simla, of 1904—Paper No. 37, On Lessons to be Learned from Sirhind Canal Silt Trouble, p. 165—Office of Supt. Govt. Printing, India, Calcutta, 1905.

The Caméré System of Fishways Installed in the Dams of the Rivers l'Hyeres and l'Aulne—Annales des Ponts et Chaussées, Part IV—1908.

See also references for Chapter I.

CHAPTER III

MAIN HEADGATES OR REGULATOR FOR CANAL SYSTEM

Object and Location.—The main object of this structure is to control and regulate the water supply admitted at the head of the canal system. Safety in design, reliability of operating mechanism, and good construction are of great importance, for the failure of the structure may admit water in the canal which may do considerable damage either to the canal system or lands below or to both.

The structure is built at the point where the water supply is diverted from the river. A diversion weir is in some cases not necessary; this condition is obtained when the low water supply of the river is comparatively large, so that it is considerably in excess of the demand, and when the diversion canal is extended upstream until its bed is brought to a depth sufficiently lower than the low water level of the stream to give a full depth of water in the canal at all stages of the stream flow. The headgates will then be located either on the bank of the main river or on a branch channel or slough, or may be at some distance back of the main river bank with an approach channel leading the water from the river to the gates. An example of a small simple structure of this type is shown by the headgate structure of the Westside Canal, Walker River Indian Reservation, Nevada (Plate VII, Fig. A).

When located on the main river bank, care must be used in the selection of the proper site, especially if the bank is made of soft friable material, such as sandy or gravelly soil, easily eroded by the currents during flood flows. When placed on a cut bank there is the danger of the erosion of the bank to such an extent that the flood flows will wash around the structure into the canal. This will require either expensive bank protection or the placing of the headgates at a distance back from the river bank, with an excavated approach channel extending in front of the gates to the river, or both. When placed on a portion of the bank not subject to cutting, there is the possibility of the deposition of

material in the comparatively still water in front of the gates and of the low-water channel of the river being formed at a considerable distance away from the bank. As a result considerable difficulty may be obtained in maintaining the proper channel toward the headgates. When the diversion is made from a branch channel or slough, the permanency of this channel must be assured; it may require either the partial diversion of the water from the main channel by a temporary or permanent diversion weir across the main channel, or the backing up of the water in the slough by the construction of a weir at its lower end. When an approach channel of considerable length is desired, some means may be necessary to scour out the sediment deposited in this channel; this may be obtained by placing waste gates in front of the headgates with a waste channel to convey the scouring flow back to the river. The capacity of the approach canal, with the waste gates shut and canal gates opened, must be at least equal to the required capacity of the main diversion canal. The sill of the escape gates must be placed sufficiently low, and the carrying capacity of the waste gates and channel must be made sufficiently large to produce a hydraulic gradient which will give the desired scouring velocity.

A diversion weir is necessary in most cases. The headgates will then be at one or both ends of the diversion weir, depending on whether one or two canals divert from the river. Where the stream water carries considerable sediment, it is important that special consideration be given to the design of the scouring sluices, according to the principles previously stated. For each canal the scouring sluices are at one end of the weir in front of the face of the headgate structure, which is parallel with the direction of flow through the sluices and usually at right angles to the axis of the weir.

In the general discussion on headworks and diversion weirs it has been indicated how the diversion weir proper may for economic and other reasons extend across the normal or low-water width of the stream channel, in which case the closure of the flood flow channel up to a height which will give a freeboard above the flood water plane on the upstream side of the weir is produced by an earth embankment at one end or at each end of the weir. This earth embankment extends from one end or each end of the weir to beyond the outer edge of the flood plane. This condition is frequently obtained when the weir site is below the

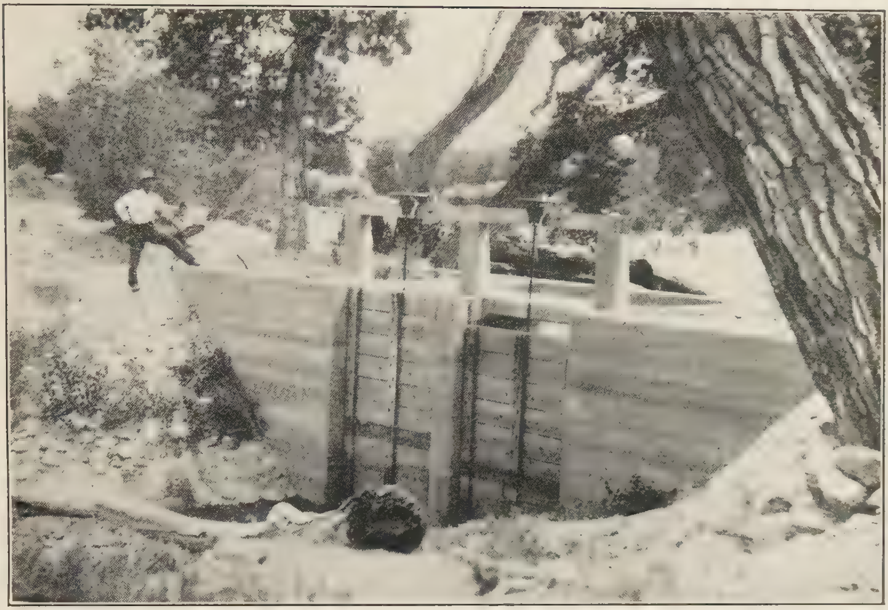

FIG. A.—Headgate of West Side Canal. Walker River Reservation, Nev.

FIG. B.—Diversion weir and headworks to two main canals on Carson River. Truckee Carson Project, Nev.

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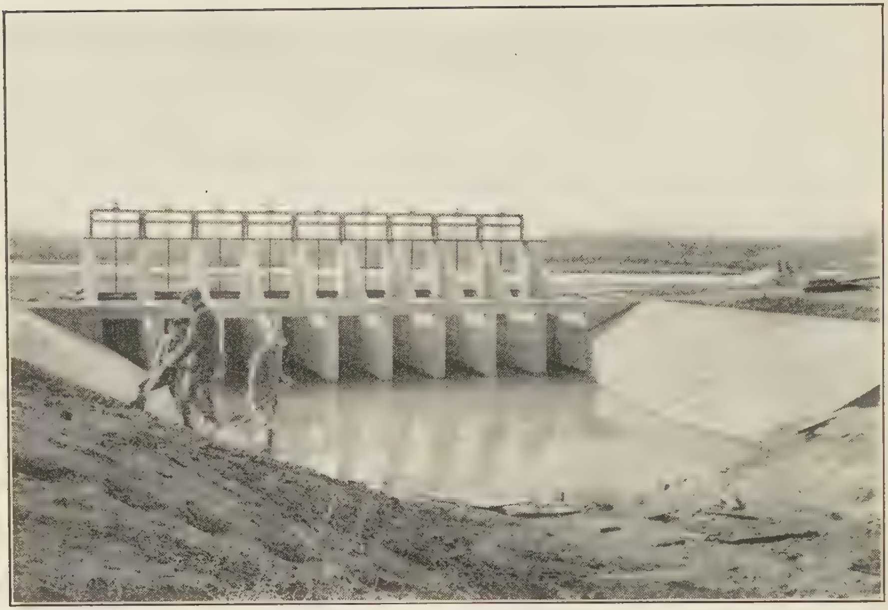


Fig. C.—Headgates at diversion works of Prewitt Reservoir Project, Colo.

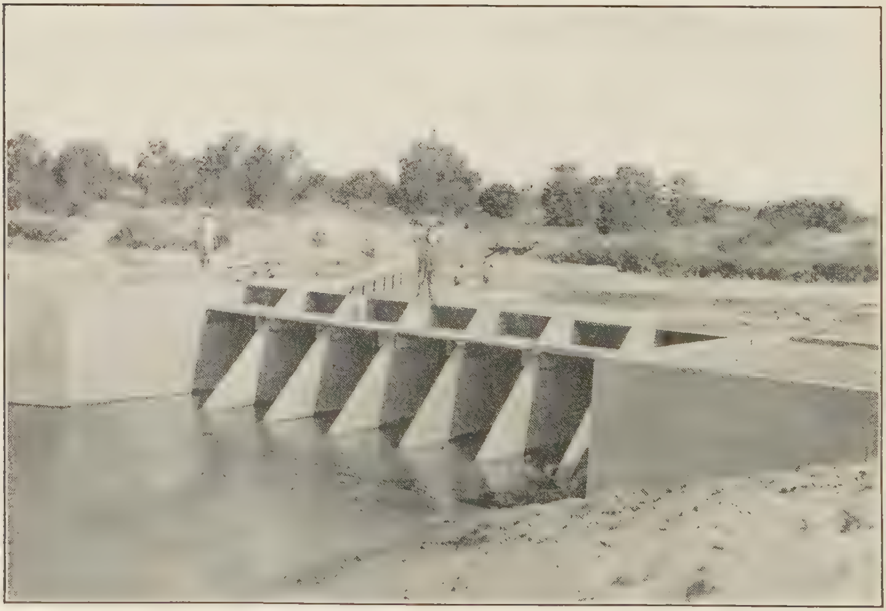


Fig. D.—Headgates and diversion works. Las Vegas Project, N. M.

point where the stream emerges from the foothill and forms a broad channel with comparatively low banks. In many cases this location will require that the canal cross section for a short distance below the headworks be partly or even wholly in fill.

The safety of the headworks and indirectly of the canal system below depends not only on the safe design and construction of the various parts, including the weir, sluices, head regulator, earth embankment and canal cross section, but on the proper juncture of these different parts and on the protection work required to maintain the stream channel and its banks.

Design of Regulator.—The factors entering in the design of the regulator are:

First.—General types of regulator.

Second.—Relative elevations of canal water surface, weir crest, canal bed, top and sill of gate openings, floor or sill of scouring sluices.

Third.—Height and width of gate openings and elevation of top of headgate structure.

Fourth.—Design of component parts of regulators.

General Types of Regulators.—The designs of regulators vary considerably, not only because of special local conditions but because of the experience and judgment of the designing engineer. However, regulators can be classified into two distinct types: the overpour or skimming type and the undershot type.

Examples of Overpour Type.—The overpour type includes such structures where the water passes from the river into the canal by discharging over the crest of a wall, or over the top edge of a gate or flashboards. Examples of this type which have been previously referred to and in most cases described in the discussion of diversion weirs and scouring sluices are the following: The headgates of the Yuma project, at the Laguna weir on the Colorado River, Arizona-Cal. (Fig. 46), the Corbett tunnel headworks of the Shoshone project, Wyoming; and headgates of the flashboard type, used on many of the older systems in the San Joaquin Valley, California, similar in form and operation to the diversion weir of the Beardsley canal on Kern River, Cal. Another example of this type, not previously described, is shown by the headworks on the Carson River, for two canals of the Truckee Carson project, Nevada (Plate VII, Fig. B). These headworks consist of a diversion weir, of practically the same

design as that used for the same project on the Truckee River, and of the headgates to two canals, one at each end of the weir. The headgate to the smaller canal is shown in Fig. 47; it forms a single opening, 15 feet wide, regulated by an overpour rising gate made of steel plate, reinforced with structural shapes. It is operated by two gate stems, one at each edge of the gate, and by two standard gate lifts, connected to a common shaft, through which the operating force is applied. The headgates to the larger canal form three similar openings, each regulated by the same size overpour gate. The gates may be operated for overpour discharge when the river carries much sediment, and may also be operated for undershot discharge at other times, to obtain closer regulation of the canal flow.

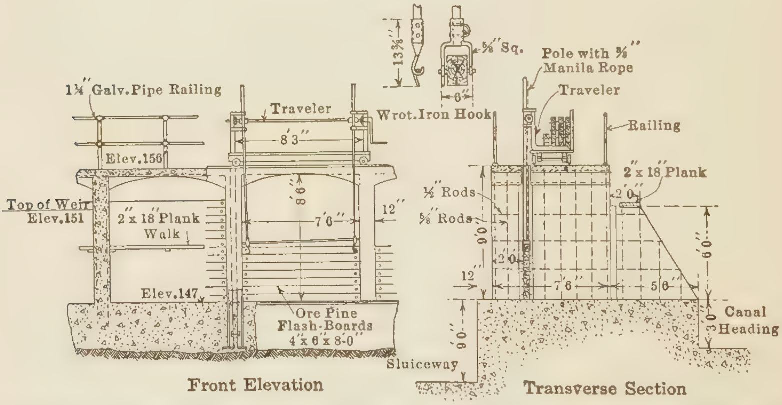


FIG. 46.—Details of headgates to main canal on California side of Yuma Project, Ariz.

Examples of Undershot Type.—The undershot type includes those designs where the water passes from the river into the canal through the gate openings formed, when raising the gates, between the sill of the gate opening and the lower edge of the gate. This type is illustrated by the following examples, previously referred to or described in the discussion of diversion weirs and scouring sluices: the headgates of the Granite Reef headworks on the Salt River, Arizona, of the Boise project, Idaho, of the Sunnyside project, Washington, and of the Truckee River diversion works, Truckee Carson project. The general form of headgate structures is also shown by the downstream view of the headgates at the diversion works of the Prewitt reservoir

project, Colorado (Plate VII, Fig. C). This structure forms nine openings, each 4 feet wide, regulated with steel gates 9 feet high, with no panel wall above the gate openings. Other examples

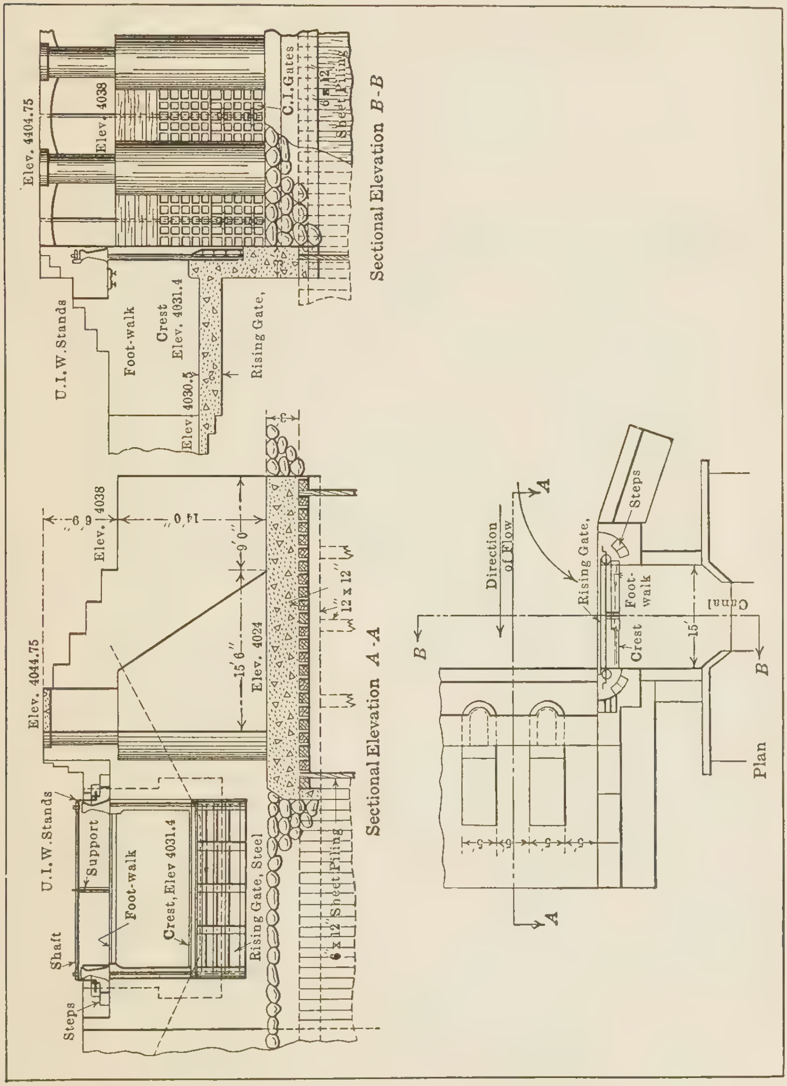


FIG. 47.—Headworks of "T" line canal on Carson River. Truckee-Carson Project, Nev.

described below are: The headgates to the Interstate Canal at the North Platte project diversion works, previously referred to; the headgates to the Las Vegas project, New Mexico (Plate

VII, Fig. D); the headgates to the Dodson North Canal, Milk River project, Montana.

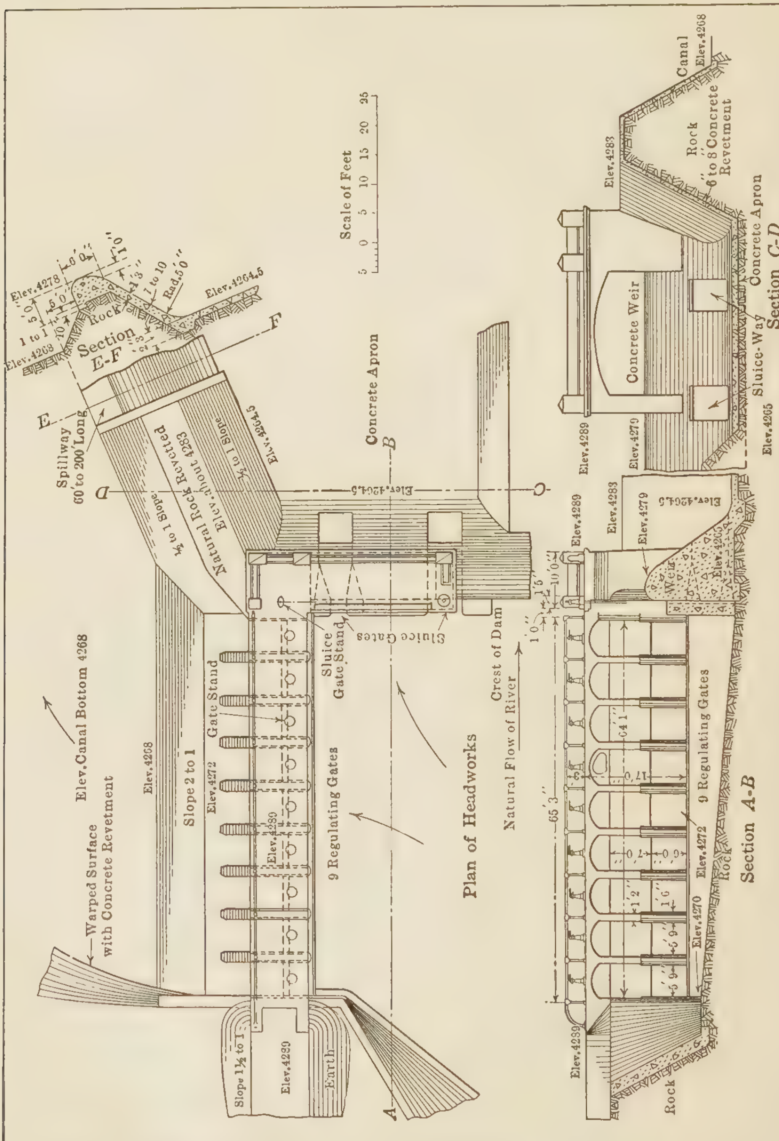


FIG. 48.—Headworks for Interstate Canal. North Platte Project, Neb.-Wyo.

The Interstate Canal headgates form nine gate openings, each 5 feet 9 inches wide by 6 feet high, separated by concrete piers,

18 inches thick, anchored with reinforcement to the floor (Fig. 48). A panel wall, 7 feet high, closes the opening between piers above the top of the gate opening. The gates are cast-iron gates, operated by standard bevel-gear lifting devices (Fig. 49).

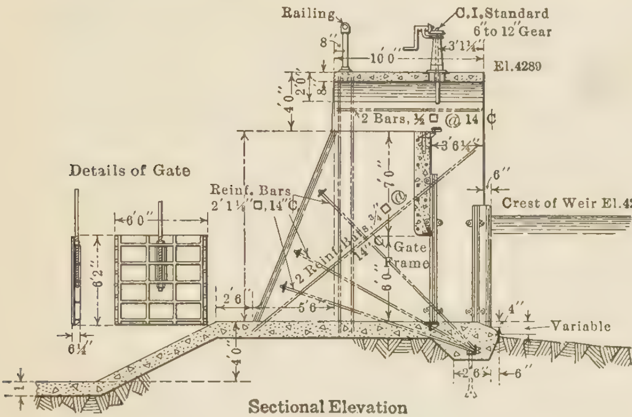


Fig. 49.—Headgates for Interstate Canal. North Platte Project, Wyo.-Neb.

The headgate structure of the Las Vegas irrigation project, New Mexico, forms eight gate openings between reinforced concrete piers, below a steel plate panel wall (Fig. 50). Each

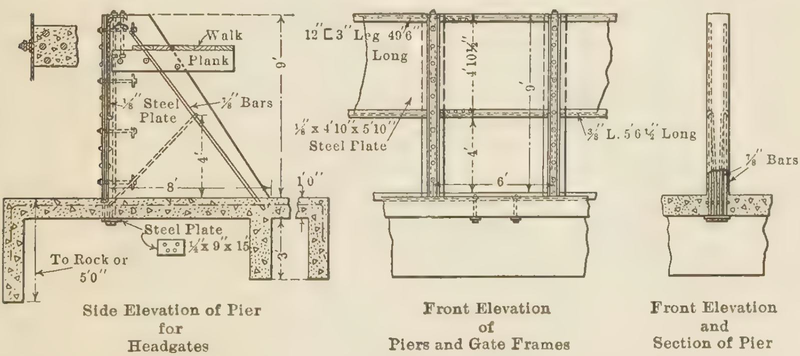


Fig. 50.—Details of headgates. Las Vegas Project, N. M.

gate opening is 4 feet high and 6 feet wide and is regulated by a steel gate operated by a rack and pinion lifting device (not shown in the diagram). The floor has three cut-off walls to increase the path of percolation, and is reinforced with wire to resist the

hydrostatic uplift pressure. The capacity of the canal is 1,000 cubic feet per second.

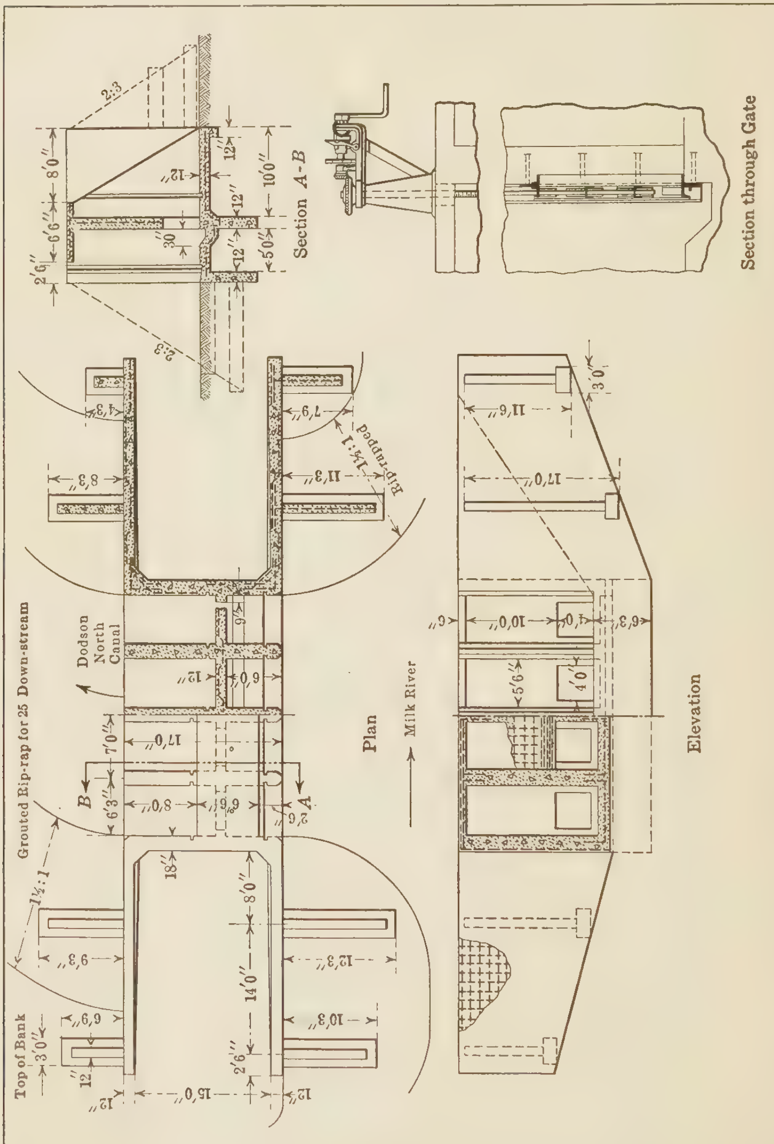


Fig. 51.—Headgates for Dodson North Canal. Milk River Project, Mont.

The headgate structure of the Dodson North Canal is located a short distance upstream from an existing diversion dam

(Fig. 51). It is set a short distance back in the river bank, and is provided with cut-off toe walls and buttressed wing walls, extending well into the earth bed and sides. The inlet is formed of the riprapped earth slopes and the outlet is protected with grouted riprap on the bed and sides of the canal for 25 feet downstream. Each of the four gate openings formed between the piers below the panel wall is 4 feet square and regulated with a gate made of ¼-inch steel plate, reinforced with channel ribs.

Comparison of Properties of Overpour and Undershot Types.—The overpour type is best adapted to regulators used in the diversion from streams whose waters carry a large amount of sediment. The best efficiency is obtained when the crest of the skimming wall can be raised or lowered to skim off the top of the river water and regulate the amount taken in the canal. A permanent crest, such as obtained by the skimming wall used for the headgates of the Shoshone project, helps to skim off the surface water, but does not regulate the flow taken in the tunnel; the regulation in this case being done with gates closing the entrance to the tunnel (Fig. 43). The simplest form of adjustable skimming crest is obtained with removable horizontal flashboards as used on the Laguna project (Fig. 46), and on canal headgates similar in design to the Beardsley diversion weir (Fig. 17).

Flashboards are always reliable, but are more inconvenient to operate than properly designed straight lift gates, and will usually give considerable leakage. Provision must be made so that the gate openings can be closed up to maximum flood water level. Where the depth of water above the sill of the gate opening is greater than 8 or 10 feet during the period of operation, the handling of flashboards involves greater difficulties and results in considerable leakage. The height of rising overpour gates, as used on the Carson River headworks (Fig. 47), is limited to the depth to which the lower edge can be lowered below the crest of the permanent sill; where greater height is necessary, each gate may be built in two or more segments which operate in separate sets of grooves, so that the segments may be lowered in front or back of each other; or the upper segments, when not needed to skim the water, may be raised toward the operating platform up to a height that the lower edges of the segments will clear the surface of the water. The upper segments can be made of greater height than the lower

segment, so that a single upper segment with the lower segment may be all that is necessary. This form of gates was used for the headgates to the Sirhind Canal (Fig. 32). The efficiency of the overpour type of headgate will depend on the length of the skimming wall.

The use of the undershot type of gate is not favorable to the prevention of entrance of sediment in the canal. This may be partly remedied by making the sill of the gate openings as high as feasible above the floor of the sluiceway or bed of the stream, using wide gate openings of small height. In many headgate structures, a set of grooves for the insertion of flashboards is provided near the upstream edge of the piers in front of the grooves for the main service undershot gates; by inserting flashboards in these grooves during the period that the stream carries the most sediment, a skimming effect may be obtained. The undershot gate has one special advantage on the overpour type, in that when the headgates are adjusted to deliver a certain canal flow, the flow will be less affected by variations in the river water level. The flow through a gate opening is usually that of a submerged orifice, and therefore is proportional to the square root of the difference in elevation between the river water surface and the canal water surface. The flow over an overpour gate is that of a weir, which in the case of a canal regulator is submerged at least during the low-water period; the flow of a submerged weir as obtained by Clemens Herschel's formula is proportional to the depth of water on the overpour crest raised to the $\frac{3}{2}$ power, multiplied by a variable coefficient, depending on the extent of the submergence, raised to the $\frac{3}{2}$ power. A consideration of the formula for flow through an orifice shows that for an increase or a decrease in the head or difference in elevation producing the flow, the percentage of variation in head is to the percentage of variation in flow as 2 is to 1; for instance, a 10 per cent. increase in head will produce a 5 per cent. increase in flow. A similar consideration of the formula for flow over a weir (not submerged) shows that the percentage variation in depth of water on the crest is to the percentage variation in flow as 2 is to 3 for instance, 10 per cent. increase in head will produce a 15 per cent. increase in flow. The advantage in regulation of flow of an undershot type of gate is greatest during the period of flood flow and decreases as the flow becomes smaller. During the low-water

period the head producing the flow through the opening of an undershot gate is small, usually less than 1 foot, and the flow through the gate opening will then be affected to a greater extent by a rise or fall in the river water surface than it would be affected by the same rise or fall during a period of greater stream flow. On the other hand, with an overpour type of gate the depth of flow on the weir corresponding to the full capacity of the canal remains the same except as it becomes submerged, when a greater depth is required.

Relative Elevations of Canal Water Surface, Weir Crest, Bed of Canal, Top and Sill of Gate Openings, Floor or Sill of Scouring Sluices.—To insure a full depth of water in the canal corresponding to the full supply carrying capacity of the canal, the water level of the river must be higher than the water level in the canal by the head or difference in water levels necessary to produce the required discharge through or over the regulating gates. The low water flow of the river after diverting the full supply canal capacity may be sufficient to give this required difference in elevation without the use of a diversion weir, but this condition is not usually obtained. In the majority of cases the low water supply is not sufficient to supply the demand; it is therefore necessary that the crest of the weir be higher than the full supply water level in the canal by the head required to produce the flow through or over the gates of the canal.

In the case of an undershot gate, the flow through the gate openings is obtained by the formula for flow through a submerged orifice, and the greater the head is, the greater will be the velocity and the smaller the required total area of gate openings; but to increase the head it is necessary to raise the weir crest, consequently it will usually be more economical to use a small head. A study of a large number of headworks constructed in the United States shows that the difference in elevation between the weir crest and the full supply water level in the canal ranges from about 6 inches to 1 foot. Assuming 0.7 as a safe value for the coefficient of discharge, the discharges per square foot of gate opening produced by a 6-inch and a 1-foot head are about 4 cubic feet per second and 5.5 cubic feet per second, respectively. In most cases it is preferable to use a head of not over 6 inches.

In the case of overpour canal gates, the flow will usually be that of a submerged weir, except possibly during high stages of the river, or if the diversion weir crest is built considerably higher

than the desired water level in the canal, which would seldom be the case. Usually the difference in elevation between the crest of the diversion weir and the full supply water level is about the same or a little larger than for the undershot gate, ranging from about 6 to 12 inches. On the headgates to the canal on the California side of the Yuma project the difference is 0.7 foot. The lowest position of the crest of overpour gates, which may be the top of a permanent raised sill, and the clear length of crest are dependent on each other, and are determined by fixing the difference in elevation between diversion weir crest and full water supply level in the canal and the volume of water to be delivered per lineal foot of gate crest. For instance, on the Yuma project, the headgates to the canal on the California side of the Colorado River give a net length of overpour crest of 262.5 feet for a required full canal capacity of 1,400 second-feet, or 5.33 cubic feet per second per lineal foot. The difference in elevation between the crest of the diversion weir and the full supply water level is 0.7 foot. With this difference in elevation and a flow of 5.33 second-feet per lineal foot, the required depth of water on the crest of the overpour gate obtained by the submerged weir formula is about 1.5 feet. This determines the highest position to which the overpour crest may be raised to deliver full canal supply during the period of low water flow when the stream flow is not sufficient to give any or only a small depth of water on the weir crest. Since in this case the stationary crest of the sill is 4 feet lower than the crest of the weir (Fig. 46), the above condition would be obtained by raising the crest with a depth of flashboards of 2.5 feet.

In the above example, the unusually large length of overflow crest with the corresponding small depth of river water diverted into the canal by skimming are desirable features, especially where as in this case the river carries an unusually large amount of sediment. The extent to which one may be justified in increasing the efficiency of the skimming effect by increasing the length of overflow crest depends also on the cost. As a guide in the determination of the length of overflow crest, two further examples are given by the headworks to the Sirhind Canal (Fig. 32) and the Jamrao Canal, diverting water from rivers carrying considerable coarse sediment, both in India and both considered successful in the prevention of entrance of coarse sediment in the canal. The Sirhind Canal regulator provides an adjustable

overpour crest 273 feet in length for a full supply canal capacity 6,000 second-feet; the Jamrao Canal regulator has an adjustable crest length of 150 feet for 3,200 second-feet. These two examples indicate a crest length of 1 lineal foot for about 20 second-feet. On this basis and assuming a difference in water levels between either the lowest position of the water surface in the river or the crest of the diversion weir and the full water supply level in the canal of 1 foot, the depth of water on the overpour regulator crest is about 5 feet, with 80 per cent. submergence.

The above considerations present the factors which determine the type of regulator, whether overpour or undershot, and the position of the crest of the weir with respect to the full supply water level in the canal. The position of the bed of the canal is determined by the design of the canal cross section. In the case of the overpour type of regulators these considerations also determine the highest position which may be given to the stationary overpour crest. It is desirable to make the stationary crest lower than this position by about 2 feet and depend on the adjustable crest, such as flashboards or raising overpour gates, to increase the height. In the case of undershot gates the openings are formed below a panel wall which extends up to the operating platform above flood water level. In order to keep the area of gate openings as high up as possible and to make use of the full gate openings, the lower edge of the panel wall or the top of the gate openings is made level with or not exceeding 6 inches lower than the full supply water surface of the canal. The position of the sill of the gate openings when the water is taken from a stream carrying considerable sediment is largely controlled by the position of the floor of the scouring sluices; it should be at least 4 feet above this floor and preferably more, and will usually be a raised sill higher than the bed of the canal.

Height and Width of Gate Openings.—In the case of the overpour type of regulator, the total crest length of gates is determined by adopting a certain volume of water to be discharged into the canal per lineal foot of overpour crest. As indicated above, a value of 20 cubic feet per second per lineal foot may be used.

In the case of the undershot gate type of regulator, the total cross-sectional area of gate openings is determined by the velocity of flow through the gate openings corresponding to the difference

in elevation between either the weir crest or minimum river water level and the full supply water level in the canal. A total area based on a discharge of 4 to 5 cubic feet per second per square foot of area is commonly used.

The top of the gate openings formed by the lower edge of the panel wall is, as stated above, at the same level as or a little lower than (not over 4 to 6 inches) the full water supply level in the canal. The bottom of the gate openings or sill will in some cases be determined largely from the position of the floor of the scouring sluices; but where conditions are obtained which do not fix the sill of the gate openings, then the selection between a low and a high sill must be made. A high sill decreases the height of gate openings and increases the total width of gate openings, but decreases the maximum intensity of pressure on the gates and is preferable when the water carries sediment. On the majority of projects in the United States the height of the undershot gates is usually from 4 to 6 feet. The total width of gateway is usually divided by piers, buttresses or columns into a number of openings. The width of each gate depends on the water pressure, the required operating force, the type of gate and the relative economy of fewer wide gates as compared to a larger number of narrow gates. Certainty of operation and simplicity of operating mechanism are of prime importance. The width is seldom greater than 8 feet and is usually from 4 to 6 feet. W. G. Bligh has criticised the comparatively narrow gates used on the majority of irrigation projects in the United States. The criticisms may be largely due to a comparison with the gates used on a few projects in India where the volumes of water diverted in the canal are many times the volume diverted through the structures which he has criticised. On the other hand, Mr. R. B. Buckley in his book on the Irrigation Works of India states that the vents of head sluices are generally 5 or 6 feet wide only, and that friction rollers are rarely fitted to the gates.

The piers, columns, or frames, which separate the gate openings, extend up to above maximum flood flow water level and support the operating platform. In the undershot type of gate the bays above the top of the gate openings are usually closed by a panel wall. In the overpour type the full openings up to the top of the structure are closed by flashboards or rising gates. The top of the headgate structure is usually about 2 feet above the maximum flood flow water level.

Design of Component Parts of Regulator.—The regulator may consist of:

(*a*) The substructure, which is formed by the floor, aprons or cut-off walls, and usually a raised gate sill.

(*b*) Side walls, which, with the floor, form the channel of the regulator; and wing walls, which form the inlet and outlet to the regulator channel and connect it with the stream bank and with the canal cross section.

(*c*) Buttresses, columns or frames, which divide the width of the channel of the regulator into a number of bays or openings closed and regulated with the gates.

(*d*) A panel wall, used only in the case of the undershot type of regulator, extending from the top of the gate openings to the top of the structure or operating platform; and the operating platform.

(*e*) The gates.

(*f*) The gate-lifting device or operating mechanism.

In addition there may be formed in front of the headgates an overflow skimming wall and basin, as previously illustrated by the headworks to the Corbett tunnel on the Shoshone River. This form is, however, very unusual, and if it be desirable, the skimming wall should preferably be provided with flashboards, which permit adjusting the depth of the overpouring sheet of water.

The design of canal headgates is dependent largely on the type adopted, whether overpour or undershot, the material of which it is constructed, the character of the material of the stream bank in which it is built, the stream flow conditions, the volume of water diverted, etc. On account of these numerous factors and the difference in judgment of the designing engineer there are and can be no standard plans, but there are principles of design which will usually control the design of each of the component parts and of the structure as a whole:

(*a*) *The design of the substructure,* of the channel formed by the floor and side walls and the design of the inlet wing walls to connect with the river bank, and of the outlet wing walls to make the connection with the canal section will depend largely on the character of material through which the regulator of the channel is formed. When built in solid rock, these parts of the structure may be entirely eliminated, and the framework for the gates is then secured to the rock sides and bed. When built in softer material, all these component parts are needed to protect against

erosion and especially to prevent failure of the entire structure by percolation under the floor or by washing around the structure. The substructure will then be built usually of concrete, although it may be built of wood; it will consist of the floor on which the piers may be built, a deep cut-off wall at the upstream end and a shorter cut-off wall at the downstream end, with in some cases intermediate cut-off walls, and in most cases a raised gate sill formed by a low wall extending above the floor between the gate piers or frames.

The floor usually extends a short distance upstream from the gate sill, at least to the edge or nose of the buttresses which project a distance above the face of the gates of from usually 2 to 6 feet to form grooves for the insertion of emergency gates or flashboards. The main object of the floor is to protect the canal bed from the erosive action of the water discharged through or over the gates and to give with the cut-off walls a sufficient length to the path of percolation under the structure. In the case of undershot gates the velocity through the gate openings will be large when the water level on the upstream side of the gate is high, as during the flood period, but the velocity is decreased rapidly within a short distance below the gates by the quieting effect of the large body of canal water. In the case of an overpour gate, the floor must be made sufficiently strong to resist the impact of the falling sheet and of sufficient length to receive the falling water well within its downstream end.

The length and thickness of the floor will be largely determined by the character of the material on which it is built. In the majority of cases the structure is built in the firm river bank; the upper part of the structure may be in the surface soil or in a made fill, but the lower part is usually in cut and the floor will often be built on firm impervious material, such as rock, shale, clay, or on more or less pervious soil, underlaid at a small depth by an impervious strata, which may be reached by a water-tight cut-off wall placed along the upstream edge of the floor. For these conditions the floor may be given a minimum length and thickness. The minimum length may be partly determined from the base width of the buttresses or frames or in the case of overpour gates may be made so as to receive the sheet of falling water about midway between the crest of overpour and the downstream end, as determined in the discussion on drops, Chapter VII. W. G. Bligh recommends for safe practice in India that the minimum length

be twice the maximum full head of water measured from the floor level to the maximum flood height. This length is larger than that used on a number of successful structures, built in the United States, where the main floor usually extends downstream only to the end of the buttresses or to a distance beyond the face of the gate equal to the height of the flood flow water level above the floor. This floor may be extended with a concrete lining or riprap for at least an equal additional length and completes the connection with the canal section. The thickness of the main floor when made of concrete ranges from about 6 to 12 inches when built on hard impermeable material, such as shale or rock, to 12 to 18 inches when on softer amterial, such as clay or clay loam. The concrete lining floor extension is usually 4 to 6 inches thick.

When built on loose permeable material, it is necessary to consider the underflow and the hydrostatic uplift pressure in accordance with the principles presented in the discussion of weirs on pervious foundations. It is seldom, however, that the river bank material will be as permeable as the material of the stream bed, nor is the floor of a headgate structure and the bed of the canal below exposed to such disturbing dynamic forces as that of the diversion weir; for these reasons a length of path of percolation equal to not over 75 per cent. of that obtained with the coefficients given for weirs on pervious foundations will be ample. The path of percolation will usually be formed by the cut-off walls and the undersurface of the floor, to which will be added in some cases the path of percolation of the sluiceway floor in front of the gates and of the floor extension either as a concrete lining or riprap. To consider the cut-off walls as forming part of the path of percolation, they must be water-tight; this may be obtained with concrete walls and special types of sheet piling, but when the sheet piling must be driven through material containing cobbles and especially with certain types of sheet piling, the water-tightness is uncertain. On pervious foundations the upstream cut-off wall is usually carried to a depth below the floor equal to the maximum flood height of the water surface above the floor, and the downstream cut-off wall at the lower end of the main floor usually extends to a depth about equal to the depth of water in the canal. Intermediate cut-off walls may be added to increase the path of percolation. The length of the floor must be sufficient to make up the remain-

der of the required length of path of percolation. A length of floor equal to about twice the height of the flood flow water level will often be sufficient, with the remainder of the path of percolation formed by a floor extension with open joints to prevent hydrostatic uplift.

The thickness and design of the floor will depend on the hydrostatic uplift pressure, as determined by the path of percolation. The floor will usually be of reinforced concrete, designed as a slab bearing up on the foot of the buttresses and side walls. The required thickness will seldom be over 18 to 24 inches.

The hydrostatic uplift pressure produces an overturning moment, which must be added to the overturning moment of the water pressure on the face of the gate when considering the stability of the structure as a whole. The counterbalancing moment is the sum of the moments of the weight of the different parts of the structure, of the downward earth pressure on horizontal surfaces or footings of walls projecting into the earth, and of the water pressure on the upstream floor extension above the gates.

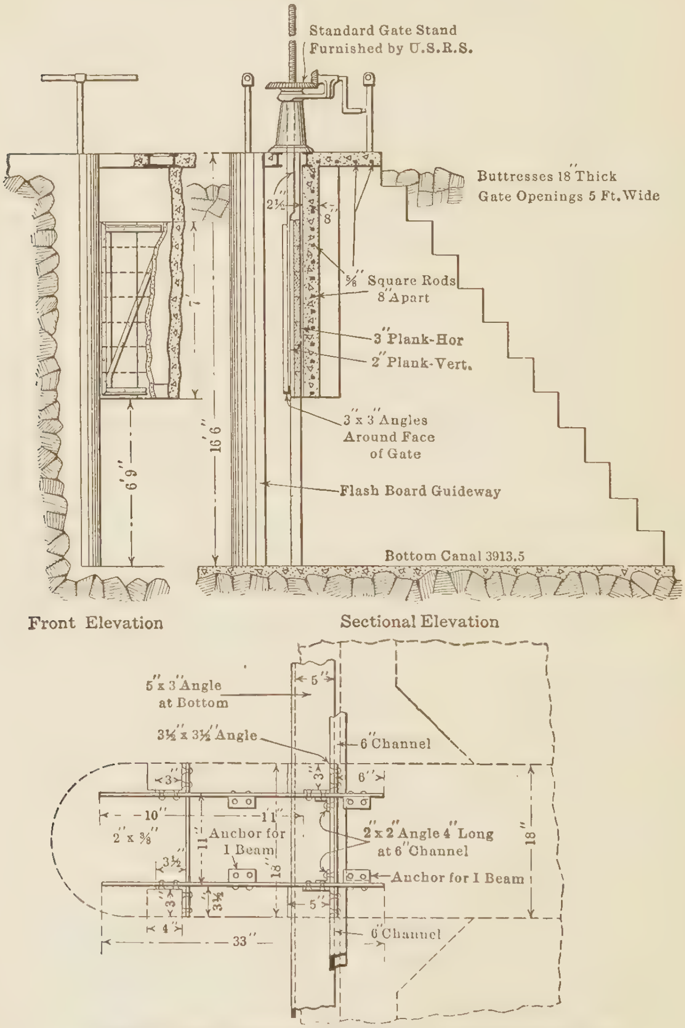
(*b*) *Side Walls and Wing Walls.*—The inlet wing walls, when the regulator is at one end of the weir with the axis of the gates at right angles to the weir, usually consists of upstream and downstream side wings, extending from the ends of the side walls, usually at right angles to them, parallel with the face of the gates and with the bank of the river, and of cut-off wings which extend from the end of each side wing well into the permanent bank of the river. The side wings protect the bank of the river for a short distance above and below the gates and form one side of the sluiceway channel when scouring sluices are provided. The downstream side wing is also the abutment which connects the diversion weir or scouring sluices with the headgate structure. When a sluiceway channel is formed by a parallel division wall in front of the headgates, the upstream side wing will extend up as far as the nose of the division wall. The downstream wing will usually extend to the downstream end of the weir floor or scouring sluice floor (Figs. 2 and 33). When no sluiceway channel is provided, the disturbing effect produced by the diversion of water through the headgates or by the discharge through scouring gates is confined to a small distance from the gates, and the upstream side wing need extend only a sufficient distance to hold a good wedge of earth between it, the side wall and cut-off wall. The cut-off wings connect the end of the side

wings with the bank of the river and are intended to prevent the water from washing around the structure. They extend from the ends of the side wings, into the permanent bank of the river, on an angle of 45° to 90°, for a length equal to at least the maximum depth of water in the stream or sufficient to connect with the sloping bank of the river, which in the case of erodable material is usually paved with riprap, brush revetment or concrete lining, for a small distance adjacent to the upstream wing and for a much larger distance beyond the end of the downstream wing (Fig. 33). When the regulator is built in a cut at some distance from the bank of the river, the side wing walls will extend into the sides of the cut and will act also as cut-off walls (Plate VII, Fig. A, and Fig. 51).

The side walls with the floor in between form the channel of the regulator. If built in porous material, they must be designed to resist not only the earth pressure but also the hydrostatic pressure produced by the water percolating along the path of percolation around the wings. This pressure will be greatest on the upstream side wall. The outlet wing walls extend from the downstream end of the side walls and are used to connect the rectangular channel of the regulator with the trapezoidal canal section. They may be vertical wings placed on an angle to the canal axis or warped surfaces (Figs. 3 and 33).

(*c*) *Buttresses, Frames or Columns.*—Buttresses are made of masonry, concrete or reinforced concrete. They are usually trapezoidal walls, extending from the floor to the operating platform, with the space in between above the gate opening, in the usual case of undershot gates, closed by the panel wall. The top length of each buttress is usually from 4 to 6 feet or the width of the operating platform. A masonry or concrete buttress is a gravity wall, designed to produce no tension at the upstream toe; this is obtained when the resultant of the water pressure with the weight of the buttress, the gate, one section of the panel wall and of the operating platform falls within the middle third of the base of the buttress (Figs. 49 and 52). The thickness of the buttress is usually 18 to 24 inches or about $\frac{3}{10}$ of the width of the gate opening. Reinforced concrete buttresses are thinner and designed as cantilever walls, with sufficient vertical reinforcement extending from the buttress into the floor to obtain a good bond, especially along the upstream toe of the buttress (Fig. 50). The thickness is generally not over 12 inches.

Frames of wood or steel have little weight and must be rigidly connected to the floor to resist the overturning moment. A frame of wood or steel consists of a vertical post with one or more



Details of nose of pier showing built up metal grooves for gates.

Fig. 52.—Headgate at diversion works of Rio Grande Project, N. M.

inclined supports extending from the post to the floor, with additional bracing to complete the framework and support the operating platform. The post is built up to form grooves for the

gates, and is designed as a beam divided into spans by the supports. This form of construction is illustrated by the old headgates of the Consolidated Canal in Salt River Valley, Arizona (Fig. 53). The canal has a capacity of 1,000 cubic feet per second; the greatest depth of water on the gates has been 20 feet.

When columns or vertical beams are used to separate the gate openings, the columns are designed as beams with their lower points of support at the floor and the upper points at the operating platform, which is designed as a large horizontal beam or girder spanning the total width of the regulator. This form of design was used for the canal headgates of the diversion works of the Truckee Carson project on the Truckee River (Fig. 3). It will compare favorably as regards economy when the width between side walls is not too great and when it can be used in the place of high buttresses.

The grooves for gates set in concrete buttresses are formed either of channels or of built-up structural shapes, such as a combination of angles, imbedded in the concrete (Fig. 52), or of special channel castings with outside projections to give good anchorage. The grooves for metal posts are usually formed by a combination of angles or other structural shapes bolted together. For wooden posts the same combination may be used, or they may be formed of wooden pieces bolted together (Fig. 53).

In concrete buttresses it is common practice to use two sets of grooves, one set being formed near the upstream nose of the buttress, usually from 2 or 6 feet upstream from the set used for the main service gates. This upper set is used for flash-board closure in case of emergency and need not be lined with metal. The flashboards can also be used to form a skimming wall during the period of flood flow when much silt is carried, in which case the grooves should be at least from 4 to 6 feet above the lower set. The upstream edge of the buttress is usually either a rounded or a pointed nose formed by two arcs, each described with a radius equal to the thickness of the pier.

The overturning moment produced by the water pressure on the upstream face of the structure, in the case of concrete buttresses of the gravity type, is balanced by the weight of the structure, but in the lighter form of construction, with reinforced concrete buttresses, wooden or steel frames, the weight will not usually be sufficient to balance this overturning moment.

When the floor is on solid rock, the balance of the overturning moment, transferred by proper anchorage and connection to the floor, may be partly overcome by securing a good bond between the floor and foundation. For a concrete floor the adhesion between the concrete and the rock will have considerable resistance. For a wooden floor the bond is obtained by means of drift anchor bolts extending in the rock (Fig. 53). When the resisting moment obtained by these means is not sufficient, or when the floor is built on a loose foundation to which it cannot be secured, the weight of the floor must be sufficient to resist the overturning moment produced by this pressure on the gates in addition to that produced by the hydrostatic uplift. The floor may be extended upstream from the gates a distance which will produce an additional resisting moment due to the weight of the water on the upstream floor extension. On pervious foundations the hydrostatic uplift pressure on this floor extension will tend to neutralize the downward pressure. But with sheet piling or a cut-off wall along the upstream edge of the floor extension, the uplift pressure will be considerably less than the downward water pressure. The length of this upstream floor extension is commonly made from $\frac{1}{6}$ to $\frac{1}{4}$ of the full depth of water on it (Figs. 49, 50, 51).

(*d*) *Panel Wall and Operating Platform.*—The panel wall may be made of timber planks (Fig. 53), of steel (Fig. 50), or of reinforced concrete (Figs. 49, 51). It is designed as a slab supported against the buttresses, frames or columns which divide the regulator channel into panels. In the overpour type of gate the panel wall is usually omitted and the closure made entirely with the gates, or the panel wall may extend only from the normal flood flow water level up to the maximum flood flow water level.

The operating platform, usually 4 to 6 feet wide, is supported on the top of buttresses or frames and connects with the top of the panel wall. It is usually designed as a slab of reinforced concrete or wood, or as a series of small arches between the buttresses (Fig. 48). It often supports the gate stand and operating mechanism and must then be designed to resist not only the bending moment produced by the force required to open the gate, but also that produced by the force required to close the gate; with the usual type of gate these two forces will be about equal and the gate stands must be strongly anchored

to the operating platform to withstand the upward reaction on it when closing the gate. Unless provision is made in the

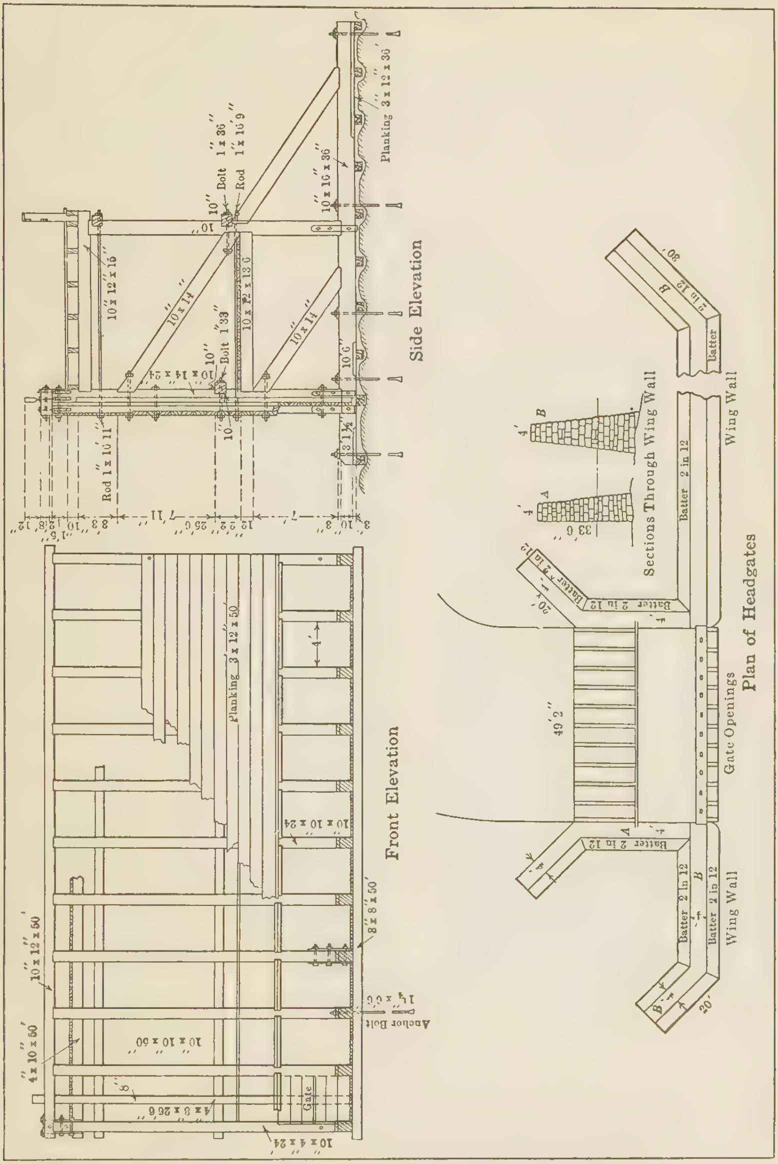


FIG. 53.—Headgates of Consolidated Canal. Salt River Valley, Ariz. (U. S. Dept. of Agr. Bull. 131. Office of Experiment Stations.)

design of the buttress and grooves for insertion and removal of the gates sideways, it is necessary to leave a narrow opening

or slot in the operating platform in line with and above the grooves through which the gates can be lowered in position. This is also necessary when the top of the gates must be raised above

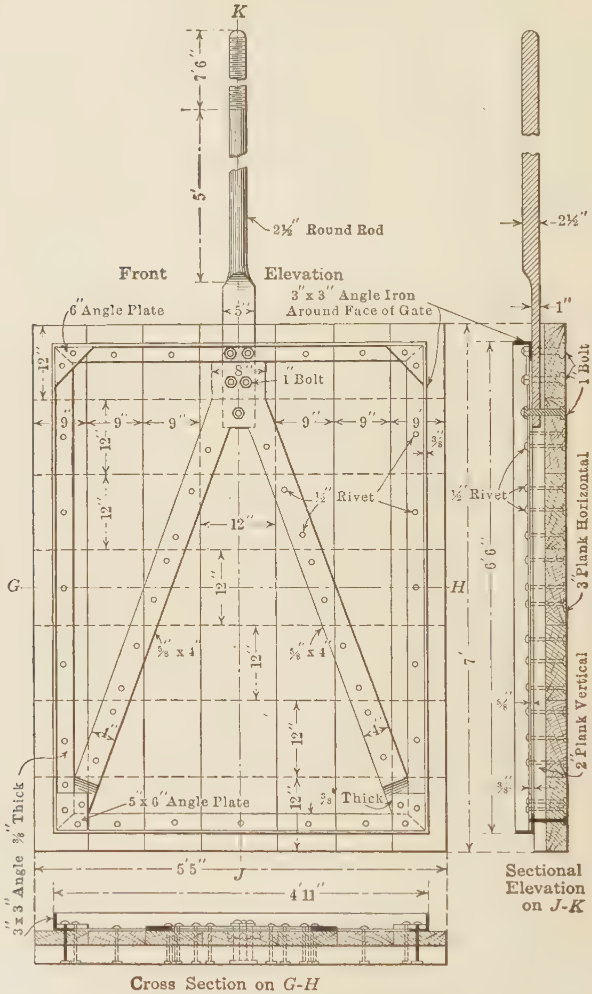


Fig. 54.—Large wooden gate for headgates and sluiceway of Rio Grande Project, N. M.

the operating platform. With certain types of lifting devices it is necessary to build a framework or extend an operating wall above the operating platform to which the device can be fastened.

(*e*) *Gates.*—The gates may be classified according to the

material used in their construction, the form of the gate, and the method of operation. The materials commonly used are wood, steel and cast iron. The most common form of gate is square or rectangular. Small wooden gates where the pressure is

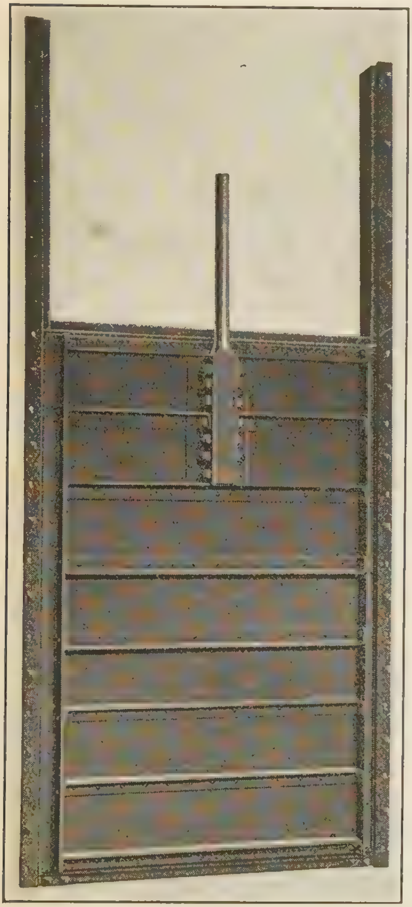


FIG. 55.—5 × 7½ ft. steel gate for the Central Oregon Irrigation Co. Deschutes, Oregon. (C. D. Butchart Co., Denver, Colo.)

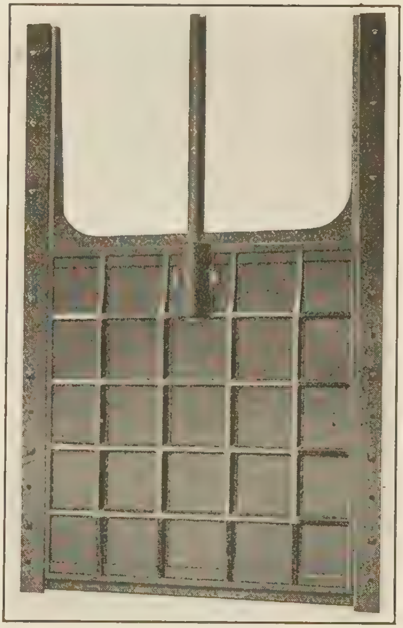


FIG. 56.—48 × 48-in. cast iron gate; medium pressure pattern. (C. D. Butchart Mfg. Co., Denver, Colo.)

not large are made of one thickness of horizontal planks at least 2 inches thick, nailed or bolted on the central axis of the gate to the vertical gate stem and usually to two additional vertical pieces, one on each side near the ends of the planks. These vertical

pieces are either wood or steel angles. The edges of the planks are planed to fit closely against each other, with in some cases battens on one side of the joints, or may be tongue-and-grooved, or grooved and joined with a spline, or shiplapped. Greater stiffness may be given to the gate by nailing the planks to additional diagonal pieces. Larger gates such as commonly used for canal headgates may be made of two thicknesses of planks; one with horizontal joints and the other with vertical joints and the side edges may be faced with metal to form a smooth bearing surface with the groove. The gate stem if of metal may be forked to run diagonally toward the lower corners of the gate (Fig. 54). Bolts are preferable to nails for all except very small gates. The thickness of the planks and the strength required must be proportioned according to the water pressure. Steel gates are made of sheet steel, usually reinforced with angles or other structural shapes (Fig. 55). Cast-iron gates consist of the cast-iron web with horizontal and vertical ribs (Fig. 56). The bearing surfaces on the side edges of large metal gates may be lined with bronze plates to give a smoother and better wearing bearing surface.

Wood gates have the advantage of cheapness and ease of construction, but are not as durable and water-tight as steel or cast-iron gates; they are not commonly used in the construction of modern permanent headgate structures. Steel gates are usually cheaper than cast-iron gates; they are best adapted to large gates of special design, but when not operated at frequent intervals are liable to cause trouble by rusting in the grooves. Cast-iron gates are more durable than steel gates and have probably been more commonly used on the majority of modern headwork structures. Cast-iron and sheet-metal gates specially designed for use in irrigation work are now made by a number of reliable manufacturers in the Western States.

The rectangular or square straight lift gate, above described, is the type most commonly used; other types used in practice are the horizontal flashboard and the Taintor or radial gate. Flashboards are planks 6 to 8 inches wide and of thickness sufficient to resist the water pressure. They are placed horizontally in the grooves, one at a time, and removed by means of hooks. When the depth of water exceeds 4 to 6 feet, they are inconvenient to handle and allow considerable leakage. For these reasons they are not satisfactory for use in important

headgate structures, although they have a very good advantage in that they are the most simple type of overpour gate. Flashboards are extensively used for the structures of the distribution system.

Taintor gates consist of a cylindrical surface or shell revolving

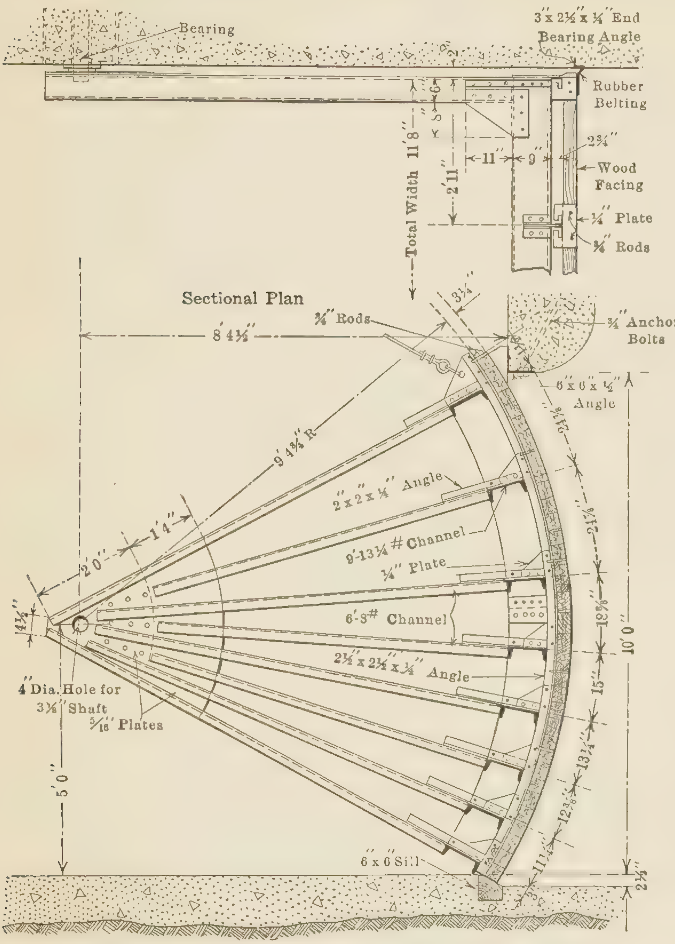


Fig. 57.—Ten-foot radial gate for canal headworks. Upper Salmon Falls Power Plant, Idaho.

around a horizontal shaft or pins to which it is connected by means of braces and radial arms (Fig. 57). The curved surface may be made of wood staves, of reinforced concrete, or of steel; the framework is made of wood or of structural shapes. The up-

stream face is convex to the water surface and the water pressure is transmitted through the framework to the pivoting shaft or axles. Water-tightness along the sides, between the side ends of the gate and the adjacent faces of the buttresses, can be practically obtained with rubber belting or similar staunching material, secured along one edge to the side ends of the gate and bent upstream to bear against the face of buttress. Water-tightness along the bottom is often secured by shaping the lower edge of the gate to bear on a wooden sill in the floor. When the upper edge of the gate does not extend above the upstream high-water level, then the opening above must be closed by a panel wall (Fig. 57). It has been found difficult in practice to obtain water-tightness along the joint between the panel wall and the gate, and serious trouble has occurred in some cases by drift material wedging itself between the lower edge of the panel wall and the face of the gate.

Since the face of the gate is cylindrical and the bearing axle is on the central axis of the cylinder, the line of the resultant water pressure, for any stage of the water level, passes through the axle and produces no tendency for the gate to revolve either clockwise or counterclockwise. The only forces to be overcome to lift the gate are the weight, the friction on the pivoting axle, and the small frictional force produced by the pressure of the rubber belting secured to the side ends of the gate on the faces of the buttresses. There is no uniform practice regarding the proportions of the gate; the average of several examples gives a radius of gate equal to about 1 to 1.5 times the height of the gate; the bearing points may be on a continuous shaft, in which case the shaft must be placed above the full supply water level in the canal in order not to catch floating material, or the bearings may be on short axles or pins firmly embedded in the buttresses. The main advantage of this type of gate is that it requires a small lifting force, which permits its use for wide openings. On account of the difficulties stated above, this type of gate has been used in only a few cases in headgate structures where a panel wall closes the opening above the upper edge of the gate. The gate is well adapted for use in escape or wasteway structures, in sandbox structures, and for the headgates to main laterals. The gate is not well adapted for structures where it is desired to skim the surface water, or for checkgates. An overpour gate will give closer regulation of the canal flow, unless it be

counterbalanced and operated automatically by floats, as is successfully done on a number of projects in California.

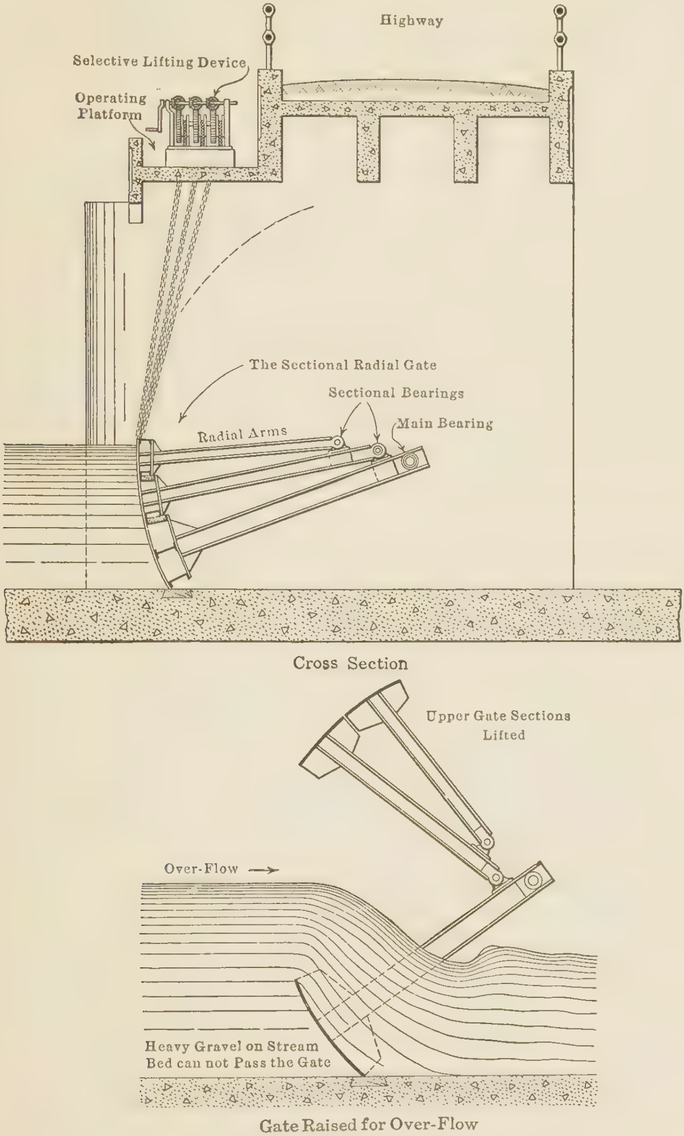


FIG. 58.—Hall segmental radial gate. (Ambursen Co., New York.)

In order to obtain the advantages of an overpour type of gate, especially the skimming effect, a segmental radial gate has been

devised by Newton L. Hall of Denver, Colorado. This gate maintains the general form of the Taintor radial gate, and differs only in that the face is divided into horizontal segments, each connected by radial arms to independent bearings (Fig. 58).

GATE-LIFTING DEVICES

The operation of a gate requires the application of a force which will overcome the weight of the gate and frictional resistance produced by the pressure on the gates. In the usual type of gate the opening force and the closing force will be nearly the same; the difference is only the weight of the gate. Except with certain types of gate, such as Stoney roller-bearing gates and Taintor gates, the weight will not be sufficient to overcome the frictional resistance. The gate-lifting device must usually be designed and secured to the operating platform or gate structure to exert a closing force as well as an opening force.

The starting force is dependent on the coefficient of static friction on the gate bearings. The coefficient of kinetic friction is less and decreases with an increase in speed of operation. The only measurements of the coefficient of static friction for canal head-gates known to the writer is a single set of measurements made by A. L. Harris on the Salt River project, Arizona. The gate was 5 × 7 feet, of cast iron with machined bronze bearing strips. The head of water on the center of the pressure area was 6 feet. The coefficient of starting friction after the gate had been closed for several weeks was 0.625.

The coefficients of static friction commonly given in text-books are the following, compiled by Rankine:

Coefficients of Static Friction

Timber on stone	about 0.4
Iron on stone	0.7 to 0.3
Timber on timber	0.5 to 0.2
Metals on metals	0.25 to 0.15
Timber on metals	0.6 to 0.2

That these values must be applied with caution to obtain starting gate friction is indicated by the measurements stated above. The high coefficient of friction obtained by Mr. Harris was probably due to the turbid condition of the water; while this condition was more serious than on most streams, nevertheless

the value obtained should carry considerable weight in selecting coefficients of friction greater than those given in the above table.

The gate-lifting devices may be classified according to the principle on which their action depends. They are included in four general classes of machine: (1) the lever; (2) the inclined plane; (3) the cord or pulley; (4) a combination of two or more of the above classes of machine. Only the simpler types of devices, operated by hand, will be considered; these include the following:

I. Lever class:

Simple lever.

Rack and pinion with operating arm.

Rack and pinion with multiple gears and operating arm.

Wheel and axle or windlass hoist.

II. Inclined plane class:

Threaded or screw gate stem and operating wheel.

Threaded gate stem, main geared wheel with worm and operating arm.

III. Pulley class:

Simple pulley.

Multiple pulley.

Differential pulley.

IV. Combined class:

Screw stem, main bevel-geared wheel, and bevel-geared pinion, with operating arm.

The majority of these devices can be obtained from manufacturers who make a specialty of the construction of gates and gate lifts for irrigation structures.

The discussion of these devices, presented below, is therefore confined to a brief description of their fundamental parts, of their efficiency, and of the relation of the operating force to the lifting force required. The detailed design of these devices involves the principles of machine design, for which the reader if interested is referred to standard books on this subject.

The efficiency of these devices requires a consideration of the efficiencies of the component parts, for which only meager data are available. This is especially due to the fact that the data are largely obtained from machines of higher grade used for different purposes and under more favorable conditions; for this reason the efficiencies selected are lower than the values commonly used in machine design.

The relation between the operating force and the lifting force is obtained from the work equation, in which the work corresponding to the operating force, multiplied by the efficiency, is equal to the work performed by the lifting force.

I. LEVER CLASS OF GATE-LIFTING DEVICE

The simple lever is used for smaller structures where a large mechanical advantage is not required. Two fulcrums are usually necessary, one for closing the gate and the other to open it. The fulcrums are usually made of a metal plate or bar bolted to a horizontal beam, placed near the gate stem, and supported about 3 feet above the operating floor. For a wooden structure this beam is usually supported on the top of the vertical posts, which form the bearing supports and the grooves of the gates. The action of the lever produces a tendency for the gate stem to bend away from the fulcrum; this can be prevented by forming a simple roller support for the back of the stem.

The relation between the operating force and the total pull exerted on the gate is expressed by the equation:

$$F = \frac{Pr}{eR}$$

where F = the operating force.
P = total pull exerted on the gate.
r = small arm of lever.
R = large arm or operating arm of lever.
e = efficiency (about 90 per cent.).

Rack and Pinion (Fig. 59).—This device consists of a rack fastened to the gate stem, a geared pinion to transmit the force to the rack, and an operating wheel or arm rigidly connected to the same axle as the pinion. Using the same notation as above and the additional notation stated below, the work equation gives the following results:

$$F = \frac{npP}{e2\pi R} \text{ or } F = \frac{Pr}{eR}$$

where n = number of gear teeth in pinion.
p = pitch of gear teeth on rack or pinion.
r = diameter on pitch line of pinion.
R = length of operating arm.
e = efficiency of gearing and bearing, 90 per cent. for cut teeth and 85 per cent. for cast teeth.

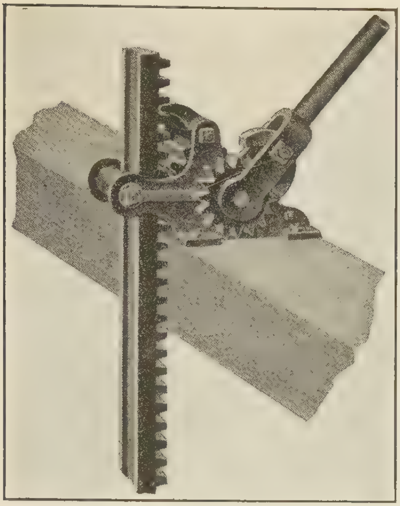


FIG. 59.—Rack and pinion lifting device. (C. D. Butchart Mfg. Co., Denver, Colo.)

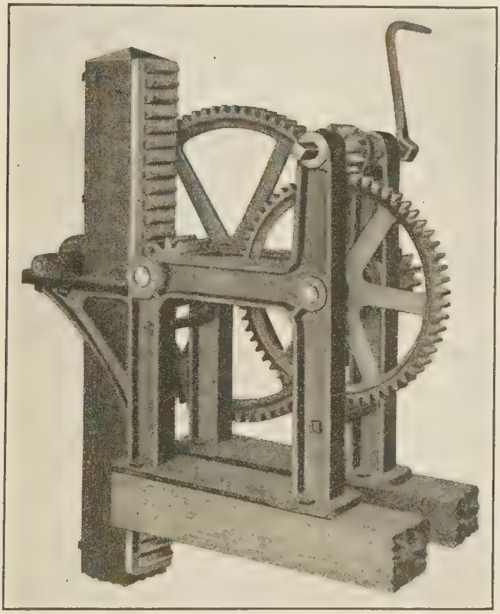


FIG. 60.—Rack and pinion multiple geared lifting device. (C. D. Butchart Mfg. Co., Denver, Colo.)

This device has a comparatively high efficiency and is well adapted when a large mechanical advantage is not required.

Rack and Pinion with Multiple Gears (Fig. 60).—This consists of the rack and pinion with additional gears to give greater mechanical efficiency. For a triple spur-geared device the relation between the operating force and the total pull is as follows:

$$F = \frac{Prr_1}{e^2RR_1}$$

where r = radius of pinion fixed on the same axle as the operating arm.
r_1 = radius of pinion acting on rack.
R = length of operating arm.
R_1 = radius of main geared wheel.
e = efficiency of gearing and bearing, 90 per cent. for cut teeth and 85 per cent. for cast teeth.

For a five-spur gear device (Fig. 60) the equation would be:

$$F = \frac{Prr_1r_2}{e^3RR_1R_2}$$

In the place of spur gear, bevel gears may be used.

Wheel and Axle, Windlass or Hoist.—A simple windlass consists of an axle to which the gate is connected by means of chains or ropes and of an operating wheel or arm, fixed to one end of the axle. Where greater mechanical advantage is required, the windlass is operated through a system of spur gears and is commonly known as a hoist (Fig. 61). It may be either stationary or placed on a traveller. When stationary, the axle is placed in line with the gate stems and may be used to operate the gates singly or in combination. The axle is usually placed at a height of about 3 feet above the operating platform. A very simple and cheap device for small gates may be made of common water pipe. This is formed of two or more pipe posts, with the lower end fastened to the operating platform or buttress walls and the upper threaded ends, extending about 3 feet above the platform floor, with tees screwed on to these upper ends to form the bearings for the axle, made of smaller size pipe, joined at one end by means of an elbow to the operating arm.

The relation between the operating force and the total lifting force is expressed by the same equations as for rack and pinion devices.

This type of device can only exert an upward pull and cannot be used where a downward force on the gate must be applied to shut the gate. Its use is practically limited to Taintor gates and roller-bearing gates of the Stoney type.

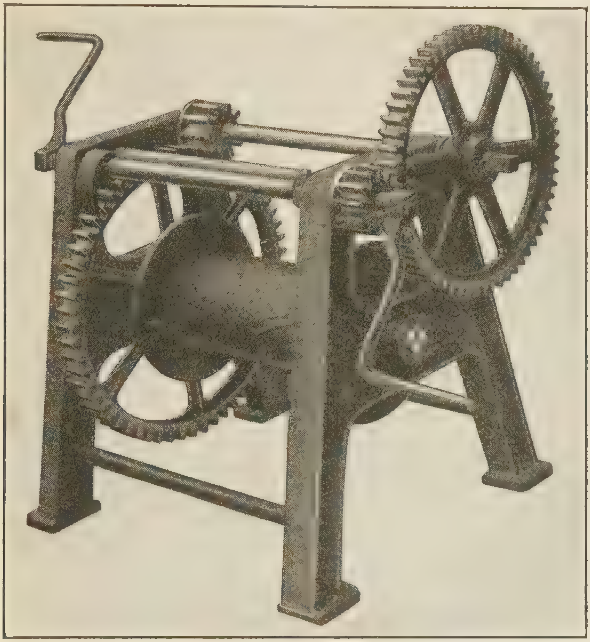


Fig. 61.—Windlass hoist. (C. D. Butchart Mfg. Co., Denver, Colo.)

II. INCLINED PLANE CLASS OF GATE-LIFTING DEVICE

Threaded Gate Stem and Operating Wheel.—This is the simplest device of this class. The lower end of the stem is usually fixed to the gate and the upper end is threaded and extends through the operating wheel (Fig. 62). The operating wheel and its bearing is either supported on a beam or bracket above the gates, or is carried on an operating stand (Fig. 63). The wheel and bearing must be held down to exert a downward force as well as an upward pull.

To decrease the bearing friction, the bearing surfaces are usually made of special metal. For this a flanged nut of phosphor bronze with threads to match the threads of the gate stem is inserted in the center of the operating wheel and keyed to it.

In the simpler type the flange of the nut bears on a brass seat, and a brass cover plate is placed over the flange and bolted to the support to hold the wheel down and form the seat for the flange when exerting a downward force on the gate. To further de-

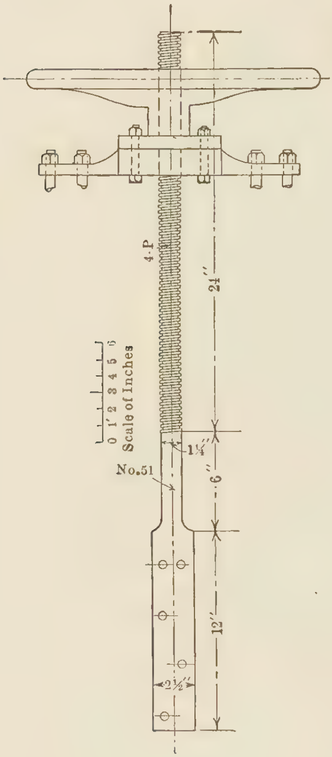


Fig. 62.—Threaded gate stem and operating wheel. (Western Steel Headgate Co., Fort Collins, Colo.)

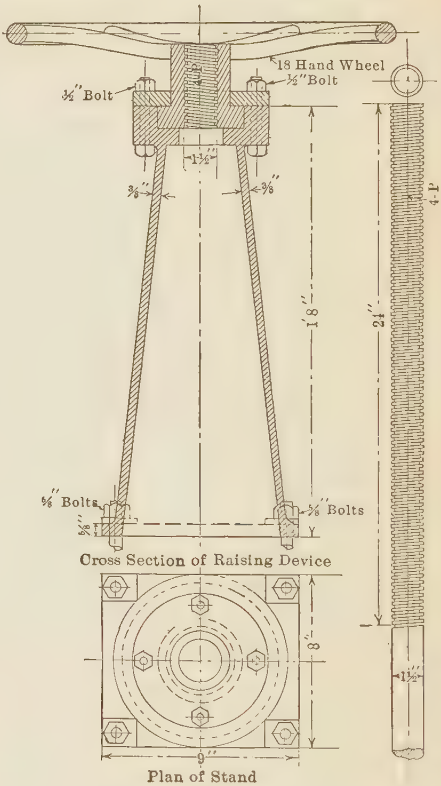


Fig. 63.—Threaded gate stem and operating wheel on stand. (Western Steel Headgate Co., Fort Collins, Colo.)

crease the friction, ball bearings are generally used for larger gates.

The gate stem may be made of a solid steel axle or of heavy pipe. It must be designed not only for tension but for torsion and compression. The minimum diameter used in practice is about 1 inch. The number of threads per inch is generally three

to four. Square threads are more efficient and are generally used. The work equation gives the following results:

$$F = \frac{pP}{e2\pi R}$$

where p = pitch of threads.
R = radius of operating wheel.
e = efficiency of the screw and seat, about 15 per cent. with square threads and 10 per cent. for V-threads.

As compared with the simple rack and pinion device, a large mechanical advantage is obtained; consequently it requires a comparatively small operating force, but on account of the low efficiency requires more work.

Threaded Gate Stem, Main Geared Wheel, and Worm (Fig. 64).—This device is used where a more powerful machine is wanted. It consists of the gate stem, the main geared worm wheel which has the same relation to the gate stem as the operating wheel described above, the worm or endless screw which acts on the worm threads of the wheel, and an operating arm connected to the worm. The parts are assembled on a cast-iron stand, which is bolted to the operating platform or gate structure. One revolution of the operating arm gives a revolution to the worm which moves the main geared wheel a distance equal to the pitch of the threads of the worm; thus a very large mechanical advantage is obtained. The relation between the operating force and the total pull on the gate is given by the following equation:

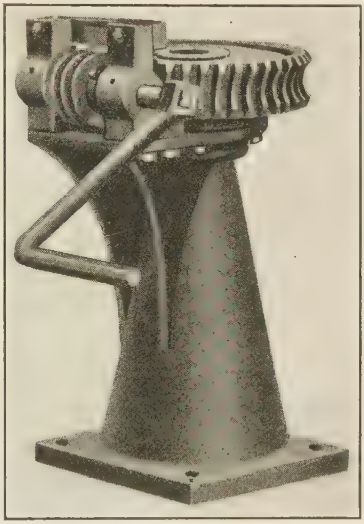


Fig. 64.—Threaded gate stem, main geared wheel and worm lifting device. (C. D. Butchart Mfg. Co., Denver, Colo.)

$$F = \frac{Ppp_1}{e_1e_24\pi^2RR_1}$$

where p = pitch of threads of gate stem.
p_1 = pitch of threads on worm or on wheel.

R = radius or length of operating arm.
R_1 = radius of pitch line on main geared wheel.
e_1 = efficiency of worm and bearings, about 40 per cent.
e_2 = efficiency of screw stem; with square threads about 15 per cent.

The efficiency selected is considerably lower than that which may be obtained in the better class of machinery. Efficiency measurements of worm transmission show that in high-grade machines, properly designed, with the worm gearing immersed in oil, the efficiency may be as high as 90 to 95 per cent. These conditions are, however, not obtained in gate-lifting machines.

On account of the low efficiency usually obtained, this device is not as well adapted for rapid operation as other devices giving the same mechanical advantage with a greater efficiency.

III. PULLEY CLASS OF GATE-LIFTING DEVICE

A simple pulley lifting device would consist of a block fixed to an overhead operating frame and of a second block fixed to the gate and connected with ropes or chains to the upper block. The blocks may have single sheaves or multiple sheaves which revolve independently. When the rope is fixed at one end to the upper block and passes in turn first from a sheave of the lower block to a sheave of the upper block and ends by passing over the last sheave of the upper block, then the relation between the downward operating force and the total pull exerted is obtained by the equation:

$$F = \frac{P}{e \times 2n}$$

where n = number of sheaves in each block.
e = efficiency, ranging from about 90 per cent. for two sheaves to 85 per cent. for four sheaves.

A differential pulley device consists of an upper block made of two sheaves of different radii, fixed rigidly to each other and on the same axis, to rotate as one piece, and of a lower single sheave block whose radius will be a mean of the radii of the upper pulley. The rims of the pulleys are shaped to hold an endless chain, which passes over the three sheaves. The relation of the operating force to the total pull is:

$$F = \frac{P}{e2R}(R - r)$$

where R = radius of large sheave of upper pulley.
r = radius of smaller sheave of upper pulley.
e = efficiency, about 30 per cent.

The simple pulley type gives a comparatively small mechanical advantage. Both types exert a pull in only one direction, and are seldom used as gate-lifting devices.

IV. COMBINED CLASS OF GATE-LIFTING DEVICE

The device of this type most commonly used combines the screw of the inclined plane class with bevel gears of the lever class, Fig.

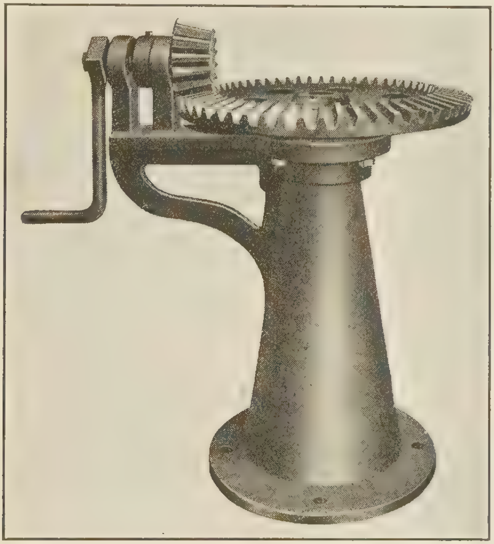


FIG. 65.—Bevel gear lifting device. (C. D. Butchart Mfg. Co., Denver, Colo.)

65. The device consists of the gate stem, a horizontal main bevel-geared wheel, a vertical bevel-geared pinion and the operating arm. The parts are assembled on a cast-iron operating stand. The work equation gives the following relation between the operating force and the total pull or force exerted:

$$F = \frac{pPr}{e_1e_2 2\pi RR_1}$$

where p = pitch of threads on gate stem.
r = radius of pitch line of pinion.

R = radius or length of operating arm.

R_1 = radius of pitch line of main bevel-geared wheel.

e_1 = efficiency of bevel gear transmission and bearing; 85 per cent. for cast teeth; 90 per cent. for cut teeth.

e_2 = efficiency of screw and bearing; 15 per cent. for square threads.

Comparison of Lifting Devices and Allowable Working Operating Force.—The selection of the type of lifting device will depend to a considerable extent on the mechanical advantage required to produce a sufficiently low operating force. This force, for single man operation, will be about 40 pounds, and the rate at which it can be applied is 150 feet per minute. When equal mechanical advantage is obtainable, devices of the lever type, such as the simple rack and pinion or the rack and pinion combined with spur or bevel gearing, are preferable to devices of the inclined plane or screw lift type on account of their higher efficiency, which reduces the amount of work to be done. When the lifting mechanism is operated by power, such as by an electric motor or gas engine, or when a large mechanical advantage is required, the screw lift, combined if necessary with worm-gear transmission, may be the best type to use. Other important considerations are the rapidity of operation, the frequency of use and cost.

REFERENCES FOR CHAPTER III

Johnston, C. T.—Masonry and Steel Headgate of the Grand Valley Irrigation Canal, Colorado—p. 140 Engineering News—August 13, 1903.

Hill, A.—Headgate of Jamrao Canal—p. 283 Proc. I. C. E., Part 3—1903–04.

Quadrant Sluices for Canal Regulators on Murrumbidgee Irrigation Project—p. 542 Engineering Record—Nov. 12, 1910.

Pueblo-Rocky Ford Irrigation Project—p. 141 Engineering Record—July 29, 1911.

Stevens, J. C.—Seros Hydroelectric Development in Spain, Part I—p. 250, Engineering Record—August 29, 1914.

Stevens, J. C.—Seros Power Development, Lerida, Spain; Diversion Dam and Canals—p. 473 Engineering News—Sept. 3, 1914.

Scobey, F. C.—Gate Structures for Irrigation Canals—U. S. Department of Agriculture Bulletin No. 115—Sept. 1, 1914.

See also references for Chapter I.

CHAPTER IV

CANAL SPILLWAYS, ESCAPES AND WASTEWAYS

Spillways and escapes are generally included in the class of structures termed wasteways. They are both used for the protection of the canal system, and are sometimes combined in one structure; for these reasons the terms are often used indiscriminately. On account of their method of operation and their action, they more properly belong to two different classes of structures.

A spillway structure acts or operates automatically and is used to prevent a rise in the canal water level above the normal full supply water level, which if not prevented would threaten the safety of the banks or cause damage by overtopping the banks. This condition may be obtained through various causes:

First.—By an excess of water entering the canal through the canal headgates.

Second.—By surface run-off or drainage water from higher lands collecting into the canal system.

Third.—By an excess flow produced by the closure of lateral headgates above.

Fourth.—By an obstruction in the canal, formed either by some material or body falling in the canal or by closing checkgates farther down in the canal.

An escape structure (Plate VIII, Figs. A, B) is generally used for two main purposes:

First.—For the protection of the canal or canal system below it in case of an emergency, by being able to divert the canal water out of the canal and prevent it from continuing downstream, and thus minimize the damage resulting from either a break in the canal, or by the washing out or failure of a structure; or to save a canal section or structure which shows sign of weakness.

Second.—For the scouring out of silt deposited in the canal section above it, in which case it may be more properly called a scouring escape, and may be combined with a sandgate structure.

The distinction between the two classes of structures is not always as marked as stated above. A spillway structure may be combined with a checkgate structure below it, to act also as an escape, in which case its capacity must be equal to the full canal capacity. On the other hand, an escape may be used to dispose of excess water, for which purpose it may be desirable to make the spillway action automatic, by using either flashboards or other type of overpour gate or by the use of an overpour crest or automatic self-regulating gates. Both classes of structure usually require a waste channel from the canal to the point of discharge into a natural drainage channel. The waste channel must usually run down a steep grade and will be either a canal with a series of drops to take up the excess fall or a chute.

Effect of Diversion of Canal Flow over Spillway or Through Escape on the Velocity and Water Depth in the Canal.—A spillway is usually designed to discharge the surplus water entering the canal, in excess of its normal or desired capacity. An escape is usually designed to discharge the full canal flow.

In many cases a checkgate structure or other means of canal closure is placed a short distance downstream from the spillway or escape structure. The necessity for this will depend on the conditions and the design.

The effect of diversion of canal flow will be considered for the case where there is no such regulation in the canal flow or no obstruction by a structure downstream from the spillway or escape. The removal of either a part flow or the entire flow of the canal will produce the same effect. In both cases it results in a decrease in the depth of water in the canal section upstream from structure with a corresponding accelerated flow toward the spillway or escape. In the case of a spillway the canal water depth decreases from the larger depth corresponding to the surplus flow capacity of the canal to the smaller depth corresponding to the capacity of the canal section below the spillway. In the case of an escape, the minimum depth of water at the structure will be determined from the design of the escape structure, as is explained farther. The drop in the water surface from the larger depth to the smaller depth produces a hydraulic water-surface gradient, steeper than the canal grade, which results in a drop down curve whose slope gradually increases to its maximum value at the spillway or escape. There is no sudden fall in the water surface; instead the decrease in

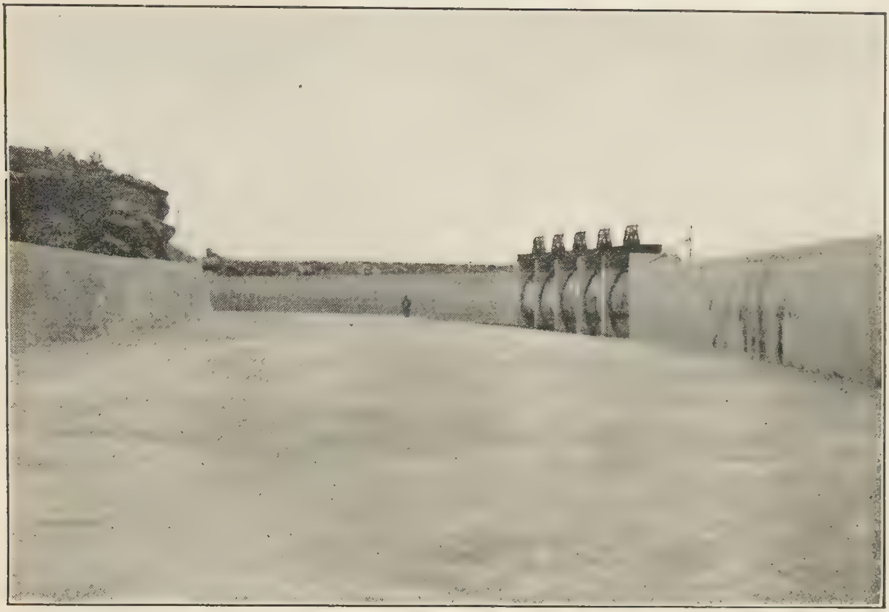


FIG. A.—Escape radial gates on main canal of Twin Falls. North Side Irrigation System, Idaho.

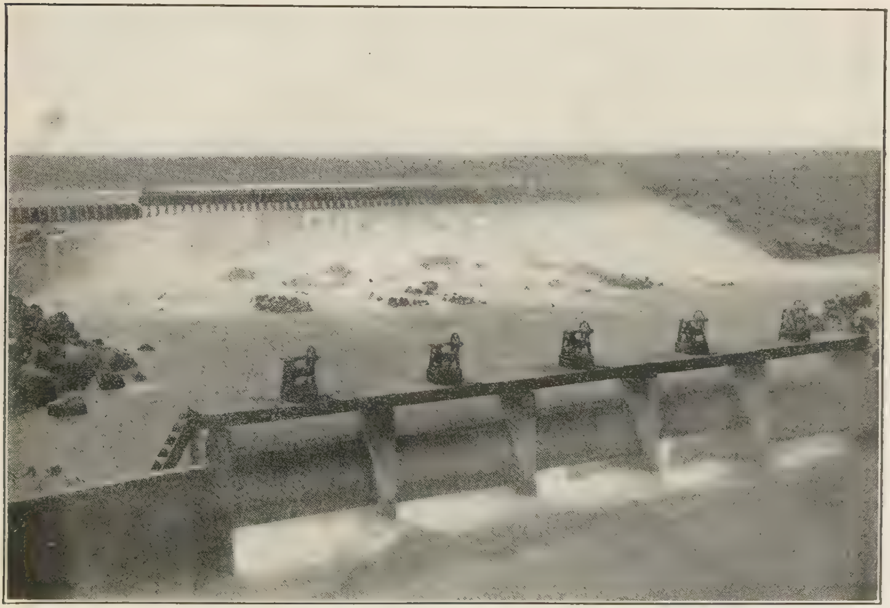


FIG. B.—Same escape gates as Fig. A, when open. Diversion dam on Snake River in the background.

(*Facing page 160*)

FIG. C.—Downstream side of spillway with wooden discharge duct. Siphonic spillway of Tennessee Power Co., Tenn. (*Eng. Rec.*, May 16, 1914.)

FIG. D.—Upstream view of siphonic spillway, showing screened openings. Siphonic spillway of Tennessee Power Co., Tenn. (*Eng. Rec., May* 16, 1914.)

water depth is gradual, beginning from a point usually several thousand feet upstream from the point of diversion. The relation between distance and drop down in the curve has been expressed by a number of formulas given in text-books on hydraulics, but they are based on certain assumptions which make the results obtained of very little value when applied to trapezoidal canals. A more accurate method is to consider the length of the canal in which the drop down occurs as subdivided into a number of sections, and for each section beginning at the smaller depth compute the rise in the water curve. These computations are made by selecting a length of section so that the drop in the water-surface curve, as represented by the difference in depth of the water at the two ends of the section, is equal to the velocity head (h_v) required to accelerate the velocity, plus the excess friction loss in head or fall (h_f) obtained from the difference between the gradient corresponding to the average accelerated velocity in the section and the smaller grade of the canal bed. The following example will illustrate the above discussion:

Surface Curve Produced by a Spillway Discharge of 150 Second-feet taken from a Gross Canal Capacity of 450 Second-feet
(Dimensions of canal given in accompanying description)

(1) Depth of water in canal in feet	(2) Velocity corresponding to depth, feet per second	(3) Hydraulic radius corresponding to depth, feet	(4) Hydraulic gradient corresponding to mean velocity and mean hydraulic radius of section	(5) Grade of canal bed	(6) h_v Velocity head to produce change in velocity	(7) hf Excess friction loss in section in feet	(8) Length of section between depths given in (1) in feet	(9) Distance upstream from spillway to depth given in (1) in feet
5.00	4.500	3.4315	....	....	....	....	....	0
			0.000823	0.00040	0.036	0.214	505	
5.25	4.233	3.561						505
			0.000706	0.00040	0.031	0.219	715	
5.50	3.991	3.690						1,220
			0.000592	0.00040	0.026	0.224	1,165	
5.75	3.771	3.817						2,385
			0.000506	0.00040	0.023	0.227	2,140	
6.00	3.571	3.941						4,525
			0.000453	0.00040	0.008	0.092	1,735	
6.10	3.496	3.991						6,260
			0.000422	0.00040	0.011	0.139	6,300	
6.25	3.388	4.064	....	....	....	....	....	12,560

Assume a canal, designed for a full supply capacity of 300 second-feet, with the following dimensions: Velocity, 3 feet per second; bottom width, 15 feet; depth of water, 5 feet; side slopes, 1 to 1, $n = 0.0225$; grade, 4 feet in 10,000 feet. Assume that the

excess water which enters the canal is 50 per cent. of its normal full supply capacity; with this excess the maximum flow in the canal is 450 second-feet, which gives a corresponding velocity in the canal of 3.38 feet per second and a depth of water of 6.25 feet. Assume that the spillway removes the excess flow of 150 second-feet, reducing the depth of water at the spillway to 5 feet, and producing a corresponding velocity just above the spillway of 4.50 feet per second. The results of the computations for the surface curve are given in the foregoing table.

CANAL SPILLWAYS

Necessity.—Canal spillways are not extensively used. On many irrigation projects either none are provided, or their use may be limited to a single one located a short distance below the main diversion canal headgates to relieve the canal of an excess supply delivered through the headgates. The necessity for them will depend on the existence of those conditions which are liable to produce an excess in flow.

A spillway is desirable or necessary:

First.—A short distance below the headgates, where the stream flow is subject to sudden fluctuations, or to daily fluctuations which will increase the amount of water discharged into the canal. This may be obtained on streams where other upper diversions are made, or where the diversion is made on a snowfed stream whose flow varies with the daily temperatures.

Second.—Where melting snows or rains occur during the irrigation season to a sufficient extent to produce a surface run-off from higher lands into the canal. This climatic condition is not often obtained and the necessity for spillways will be obviated if the drainage or run-off water collects into drainage channels which discharge either under or over the canal.

Third.—When changes in the regulation of the flow in certain parts of the system are made without making the required corresponding adjustments in other parts of the system. For instance, when there is a sudden decrease in demand for irrigation water, caused by summer rains or by the weather becoming suddenly cool, it may be desirable to close lateral headgates before the flow can be diminished by regulation at the head to the canal system. This is especially true where the point of regulation is so far away that the effect of the regulation is not felt

in the lower part of the system for several hours or as much as 2 or 3 days afterward. With a proper system of operation and with the laterals of the system continued to discharge at their lower ends into a waste channel, the excess water can be carried to the lower end of the laterals and there will be little need for spillways. It may, however, be desirable to have a spillway or combined spillway and wasteway at the lower end of the diversion line where the distribution system begins, and on large systems additional ones may be desirable at points where the system separates into main divisions.

Fourth.—A short distance upstream from specially dangerous points, such as may be obtained on diversion canals constructed on steep side hill, where the uphill material is likely to fall in the canal and partly or entirely obstruct the channel.

The necessity and location of spillways will be determined from the above considerations and also from a study of the damages which would result from an overflow of the canal banks. Where the canal is on a side hill and the canal bank made of loose material, the damages produced by an overflow would be far more serious than for a canal built in comparatively level country with the cross section largely in cut. The location of the spillways will also depend on the availability of a natural channel. It must be near such a channel in order that the length and cost of the spillway channel be not excessive. These conditions of location will also usually apply to the location of escapes, and as stated above a spillway will often be constructed with the escape structure, or the escape structure, although not automatic in action, may be used to dispose of the excess water.

Capacity of Spillway.—The required capacity of the spillway depends on the source of the excess water and will be determined from:

First.—An estimate of the possible raise in the water level of the river, and the determination of the corresponding excess flow through the headgates produced by this raise.

Second.—An estimate of the surface run-off, due to rainfall or melting snows, from lands higher than the canal, occurring during the irrigation season and draining into the canal; considering also the advantages of carrying such water wherever it can be collected, either under, over, or through the canal.

Third.—Estimates of the excess flow in the canal due to the

closure of certain lateral headgates above the location of the proposed spillway.

Fourth.—Assumptions of the extent of obstructions caused by material falling in the canal and the determination of the resulting rise in the water level. In the case of complete obstruction the capacity of the spillway must be equal to the capacity of the canal.

Unless the flow down the canal is checked by an obstruction, a checkgate, or other means placed within a short distance downstream from the spillway, there is no advantage in making the spillway capacity larger than the maximum volume of surplus water which can be carried to it in the canal without overflowing the canal banks. This maximum capacity of the spillway is the difference between the normal full supply carrying capacity of the canal and its larger carrying capacity obtained when the water level of the canal raises to the absolute minimum depth below the crown of the bank, which, in the case of an emergency, may be taken with the water level nearly up to the crown of the bank. If this maximum surplus capacity of the canal is not sufficient, it will be necessary to increase the canal freeboard. The accurate determination of the absolute maximum carrying capacity of the canal requires a consideration of the drawn down curve, produced in the surface of the water by the volume taken out through the spillway.

With the flow in the canal held back or dammed up by an obstruction or a checkgate, placed a short distance downstream from the spillway, the entire canal supply may be diverted to pass through the spillway, in which case the spillway capacity must be equal to the full canal capacity.

Types of Spillways.—The common types of spillways are:

First.—The overflow spillway.

Second.—The siphon spillway.

Third.—The automatic gate spillway.

Overflow Spillway.—An overflow spillway consists of:

First.—An overpour bank or wall, formed by a section of canal bank, or the side of a flume, or a length of retaining wall, where the crown or overpour crest, if stationary, is made level with the normal water level in the canal.

Second.—A receiving basin in which the overpour water is collected.

Third.—A waste channel leading from the spillway receiving

basin to a point of discharge, usually in a natural drainage channel.

Action of Overflow Spillway.—When the flow over the spillway is not controlled by a checkgate at or a short distance below the downstream end of the spillway, or by similar means of regulation, then the spillway will not dispose of the entire excess water in the canal, but will produce only partial relief. In order to completely dispose of the excess water, the depth of overflow must be practically zero at the downstream end of the spillway, but as the rate inclination of the drop down surface curve toward this point is very gradual, the required overpour discharge could only be obtained with a prohibitive length of spillway. This was well illustrated by the example previously given. A consideration of the results obtained show that for practical purposes the depth of water in the canal remains the same along the entire length of the crest of the spillway and is equal to the depth of water in the canal section which continues from the downstream end of the spillway. Therefore the length of spillway will be determined:

First.—By using for the depth of overpour sheet the maximum increase in depth, corresponding to the excess in flow, which can be safely allowed to continue down the canal beyond the downstream end of the spillway.

Second.—By making the carrying capacity of the spillway not equal to the entire surplus flow, but equal to the surplus flow minus the portion of the excess flow, which is allowed to go on down the canal.

For instance, in the above example, let us assume that the depth of overpour is 4 inches; this makes the depth of water in the canal at the downstream end of the spillway 5 feet 4 inches, with a corresponding discharge capacity in the canal of 337 second-feet. The surplus to be wasted over the spillway is then 113 second-feet. Using Francis' formula for flow over a sharp crested weir, the required length of spillway crest is 175 feet.

When the flow can be controlled by a checkgate placed at or a short distance below the spillway, then the maximum depth of overpour water on the crest of the spillway which may be obtained is determined by the freeboard provided in the canal, and the capacity of the spillway may be made large enough to discharge not only the surplus water in the canal, but also a part of or the entire normal flow of the canal.

In the above discussion, it has been assumed that the crest of the spillway is stationary, in which case the crest is placed at the normal water level of the canal. In many cases it may be desirable to be able to lower the crest of the spillway or make it adjustable; in the simplest form this may be obtained by flashboard regulation. Automatic regulation is obtained with certain types of balanced gates, some of which are illustrated by examples described farther.

Examples of Overflow Spillways.—Simple types are shown in Figs. 66 and 67. Fig. 66 is a spillway designed for the Sun River project, Montana. It consists of an overflow bank with a crest 100 feet long, placed at the normal full supply water level and of a short channel which discharges the overpour water in an adjacent slough. The overpour bank and the floor and sides of the upper part of the channel are lined with 4 inches of reinforced concrete with cut-off walls at the inlet and outlet ends and inlet side edges. The lower part of the channel is lined with paving.

Fig. 67 is a standard spillway designed for the Lower Yellowstone project, Montana. It consists as in the previous example of an overpour bank 100 feet long, an adjacent receiving basin parallel to the canal, extended as a waste ditch down to a point of discharge. The overpour bank, receiving basin and waste ditch are all lined with reinforced concrete 6 inches thick.

A standard type of spillway designed by the technical section of the Reclamation Service is shown in Fig. 68. This structure differs from the preceding ones in that instead of the overpour bank an overpour reinforced concrete buttressed wall is placed in the canal bank. The crest of the wall is 2 feet below the top of the canal bank. The receiving basin connects with a concrete lined waste channel, which in any case would be designed according to the principles presented for chutes.

A special type of spillway designed for a combined structure with a culvert, on the Belle Fourche project, South Dakota, is shown in Fig. 69. The overflow spillway is formed of a vertical well or shaft oval in cross section, extending from the roof of the culvert and placed in the canal cross section, toward one of the banks, with its crest at the full supply water level. At and below the connection of the shaft and culvert the cross section of the culvert is enlarged to carry the surplus water. This same type of construction was used on a project in British Columbia

for a spillway on a concrete lined canal; it was formed of a culvert pipe, placed under the floor of the canal, and of a number of

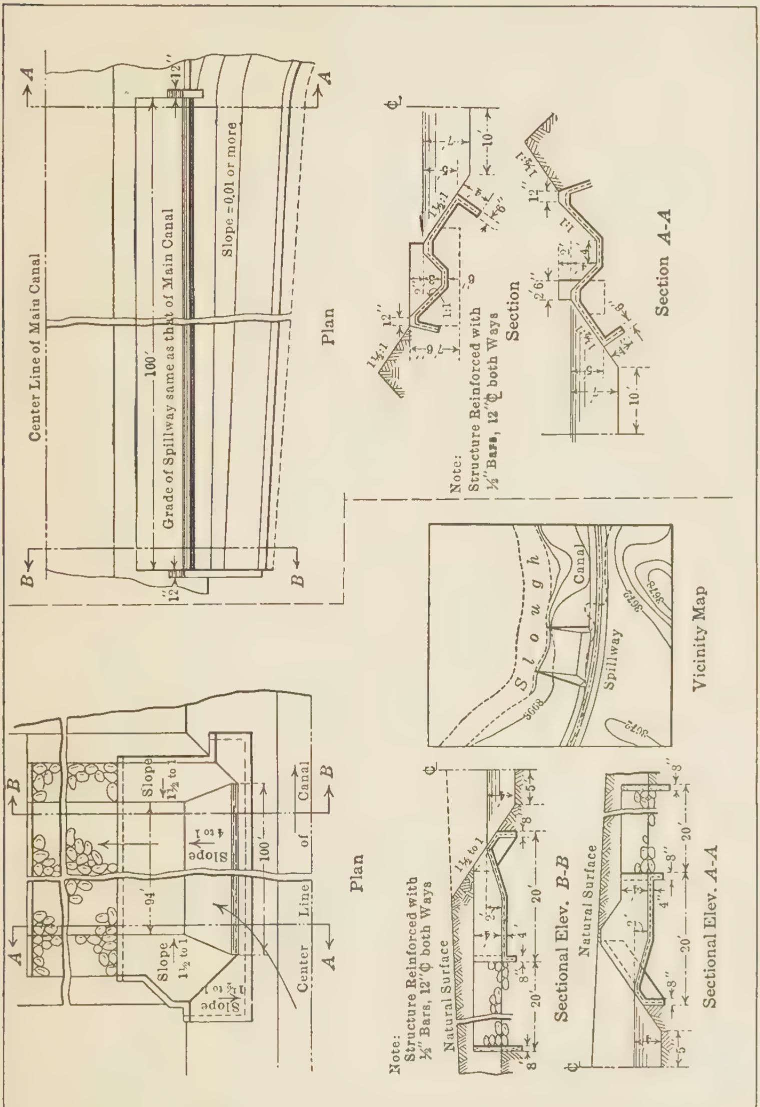


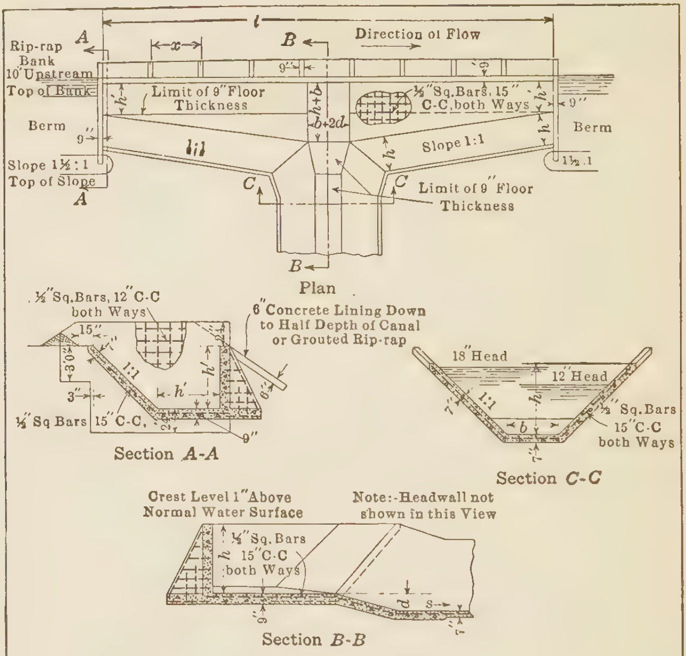
FIG. 66.—Overflow spillway. Sun River Project, Mont.

FIG. 67.—Overflow spillway. Lower Yellowstone Project, Mont.

vertical stand pipes connected at the bottom to the culvert pipe and placed in an enlarged section of the canal with their crest at full supply water level.

Other examples of overflow spillway structures are presented by combination structures used at crossings with natural drainage channels (see Chapter V).

Siphon Spillways.—Siphon overflow spillways have been used in Europe for many years on a number of power and irrigation



Dimensions							Depth of Overpour.			
							12"		18"	
l	b	h	h'	d	x	s	Q_1	V_1	Q_2	V_2
50'	3'6"	5'0"	4'5"	1.2'	8'5½"	.0020	150	6.1	305	8.0
75'	4'3"	6'0"	5'4"	1.4'	8'5"	.0018	225	6.2	460	8.6
100'	8'0"	6'0"	5'1"	1.5'	8'4¾"	.0016	300	6.3	610	8.7

Fig. 68.—Standard overflow spillway designed by U. S. Reclamation Service.

canals; most of them in Germany, Switzerland and Italy. Their use in this country is much more recent; some of the earliest examples are for a navigation canal, on the Champlain division of the New York State Barge Canal in 1910. Since then they have been used in the forebay of a hydroelectric plant of the Tennessee Power Co. (Fig. 72) and on the power canal of the

Mount Whitney Power & Electric Co. in California, and more recently the U. S. Reclamation Service has used them for an installation on the Orland project in California and for several installations on the Sun River project in Montana (Fig. 73). The structure is placed in the bank of the canal, with the throat

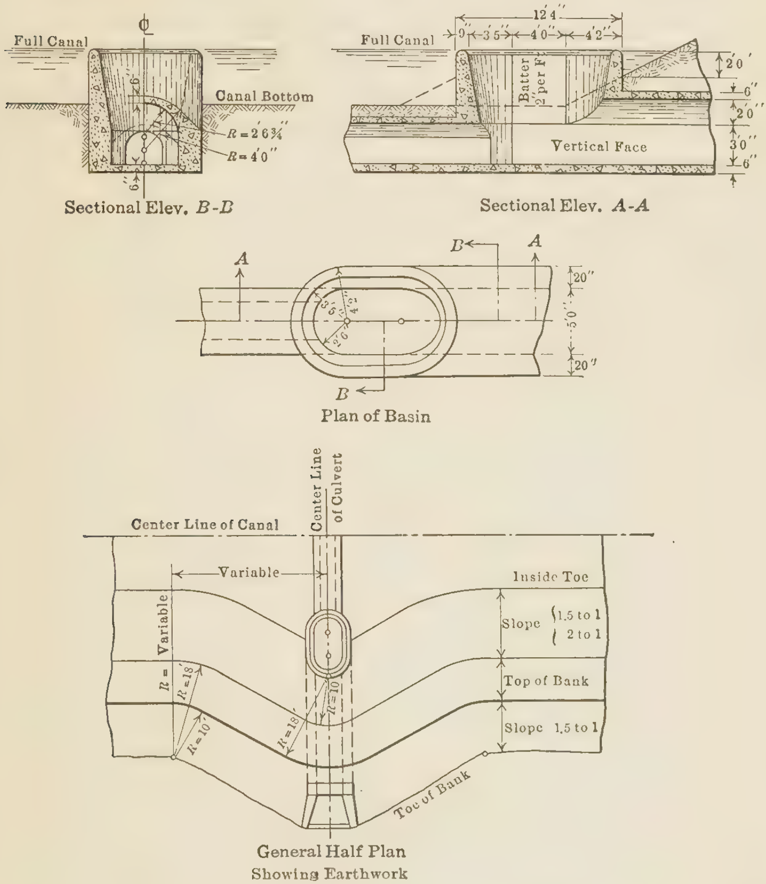


Fig. 69.—Combined overflow spillway and culvert. Belle Fourche Project, S. D.

of the siphon near the top of the bank; the short leg extends on the canal side of the bank into the water and the long leg or discharge leg is carried down the other side of the bank.

The siphon structure may consist of a single siphon unit or of a number of units. The size of each unit will depend on the feasibility of construction and on comparative cost.

Hydraulic Computations and Siphonic Action.—The flow through a siphon is produced by the difference in elevation between the water surface at the inlet and either the water surface at the outlet, in the case of a submerged outlet, or the center of the outlet opening, in the case of free discharge; this difference in elevation is the operating head.

The flow is essentially the same as that through a short tube or pipe, with various forms of inlets, with the added limitation that the velocity at the throat cannot exceed that resulting from a complete vacuum at this point. Two forms of siphon tubes may be considered: (*a*) with throat and outlet leg of equal cross sectional area, but not necessarily uniform shape; (*b*) with contracted throat area, expanding with a divergent outlet leg.

(*a*) *Computations for siphon tube, with throat and outlet leg of equal cross sectional area, but not necessarily uniform shape.*

Let:

H_0 = operating head.
H = net effective velocity head.
V_e = velocity at entrance to siphon.
V = velocity at throat and in outlet leg of siphon.
C_e = coefficient for entrance loss, usually less than 0.50, and applied to the entrance velocity (V_e).
C_i = coefficient for loss in friction in inlet leg, including also loss due to contraction in cross-sectional area. With a properly designed inlet, tapering gradually to the smaller cross section at the throat, this loss would be very small. The coefficient is probably not greater than 0.10 and is applied to the throat velocity (V).
C_b = coefficient for loss in bend which will decrease with an increase in the radius of curvature of the bend. With a radius of curvature equal to the height of the throat opening it will probably be not greater than 0.25, and is applied to the velocity at the throat (V).

The friction loss in the outlet leg may be determined from Chezy's formula, in which case, assuming the cross-sectional area of the outlet leg to remain constant and equal to the area at the throat, it is equal to $h_f = \frac{V^2}{C^2R}l$ where l = length of outlet

leg, R the hydraulic radius, and C the coefficient obtained from Kutter's formula. The net velocity head is then:

$$H = \frac{V^2}{2g} = H_0 - C_e\frac{V_e^2}{2g} - C_i\frac{V^2}{2g} - C_b\frac{V^2}{2g} - \frac{V^2}{C^2R}l$$

from which the value of V is obtained.

The use of this formula requires the selection of certain values for the coefficients from hydraulic data, which is very meager; for this reason it may be more desirable to use the simpler formula:

$$V = C\sqrt{2gH_0} \text{ or } Q = CA\sqrt{2gH_0}$$

where V = velocity throat and in outlet leg.
H_o = operating head.
A = cross-sectional area of throat and of outlet leg of siphon.
C = coefficient of velocity or discharge, which a number of experiments indicate to be between about 0.60 to 0.70.

This formula may be applied to siphons of proportions similar to those for which the coefficients were obtained, which are described further. As previously stated, these formulas can only be applied up to a fixed maximum value for the velocity. The velocity at the throat or crest of the siphon cannot exceed that produced by a perfect vacuum, corrected by the proper allowance for frictional resistance in the siphon inlet. The theoretical pressure head corresponding to a perfect vacuum is 34 feet at sea level; it diminishes to about 32.8 at 1,000 feet elevation, 31.5 at 2,000 feet elevation. In practice, on account of the air entrained in the water, the maximum suction lift is often taken as about 28 feet; the correction to be applied to this for the frictional and entrance loss in the siphon inlet depends on the form of the inlet. Using the larger values suggested above for the coefficients of entrance loss, frictional loss and loss in bend, neglecting the small head of water on the center of the throat area and assuming the entrance area is 50 per cent. larger than the throat area, the total loss in head, including ½ of the loss in the bend, is then $0.45\frac{V^2}{2g}$, which subtracted from a suction lift of 28 feet, gives a maximum throat velocity of 35 feet per second and a corresponding maximum velocity head of 19 feet. In the same way, assuming an entrance area of twice the throat area, the maximum throat

velocity is 36.5 feet per second and the maximum velocity head is 20.7 feet.

The above maximum throat velocity will only be obtained when the operating head is equal to or greater than the sum of the above maximum velocity head plus the total head loss by friction in the siphon. This frictional loss may be obtained separately, or the total operating head may be obtained by the simple formula $H_0 = \frac{1}{C^2}\frac{V^2}{2g}$. The value of H_0, corresponding to a maximum throat velocity of 35 feet per second for $C = .60$, is 53 feet, and for $C = 0.70$ is 39 feet. When the operating head is smaller than that required to obtain the maximum vacuum throat velocity, then the throat velocity is that corresponding to the operating head. When the operating head is greater there will be acceleration in velocity in the lower part of the discharge leg. Where this condition exists, the cross section of the discharge leg must not be larger than the minimum cross section at the throat, and the cross section of that part of the leg below the point where acceleration begins must be contracted to conform with the increase in velocity. The point where acceleration begins will be at a vertical depth below the inlet water level equal to the operating head required to produce the maximum obtainable vacuum throat velocity. Acceleration will continue to the outlet or down to a point where the accelerated velocity produces a frictional resistance equal to the accelerating force.

(*b*) *Computations with contracted throat area, expanding with a divergent outlet leg.*

The formula of flow $Q = CA\sqrt{2g\,H_0}$ may be used with A for the cross sectional area of either the throat or the outlet end of the divergent discharge leg, using in either case the proper coefficient of discharge, which may vary considerably with the form of the siphon and the angle or extent of divergence.

The effect of a properly expanding outlet is to increase the coefficient of throat discharge, so that it may be considerably higher than unity. In siphon construction the extent of throat contraction is limited by a maximum throat velocity, equal to that produced by a perfect vacuum. Consequently when the operating head is comparatively large, such that the outlet velocity approaches the maximum possible throat velocity, either no expansion or only a small increase in cross sectional area

should be made. With low operating heads the outlet velocity may be small and to facilitate construction it may be desirable to contract the throat area to such an extent that the throat velocity will approach the maximum velocity resulting from a vacuum. A properly shaped low head siphon with divergent outlet leg will probably give a coefficient of discharge for the outlet cross sectional area of 0.65 to 0.75 and for the throat area of 1.25 to 1.50. A study of the coefficient of discharges for compound divergent tubes as given in text-books on Hydraulics will be helpful in obtaining a proper design.

Design of Siphon Parts.—The correct proportioning must be based on theoretical considerations, partly indicated above, and on the results obtained from a number of installations operating successfully. The overpour crest of the throat is placed on the same level as the desired normal water level in the canal.

The shorter leg or inlet must be sealed by the water as soon as the canal water level rises above its normal height; this may be obtained in two ways:

First.—The inlet to the shorter leg extends only down to the normal water level, or a short distance (not over 2 or 3 inches) below the normal water level, so that a drop in the water level below the inlet edge will let air in the siphon and break its action.

Second.—The inlet leg is extended well below the water surface, and the crown of the siphon is connected with air channels whose inlets are at the normal water level in the canal. It is desirable to have the inlet to the siphon well submerged to prevent the entrance of floating material.

The throat of the siphon is usually flattened to a wide cross section. This is desirable to facilitate the priming. To bring the siphon into action, the air inlet must be sealed and the water must rise to spill a small depth of water over the overpour crest or floor of the throat. The air in the siphon is rarefied by the sheet of overpour water, which strikes the far side of the lower siphon leg and entrains air with it. A rise in the water level of 2 or 3 inches above the overpour crest is sufficient to bring the siphon into action very quickly. It is stated that with this rise large siphons on the Canal Milan, near Verona, Italy, having a cross-sectional area of 14 square feet and a working head of 20 feet, are brought into full activity in from 1 to 2 minutes. Tests made on the siphon spillway of the Tennessee Power Co. showed the period of priming

to be from 5 to 22 seconds, depending on the rapidity of raise of the water level and that the siphons would prime when the water surface rose slightly above the top edge of the air vents, or 3 inches above the overpour crest.

To break the siphonic action quickly, a comparatively large air inlet area is necessary. To provide for this and also for a quick seal, the air inlet will be long narrow openings with a sharp upper edge, or the inlet opening may face downward with the edges on a horizontal plane level with the desired water surface.

The best practice, as indicated by experience in Europe, is to use easy curves to form the throat, to make the downstream leg not vertical but inclined in a direction away from the canal, to seal the outlet by submergence in water, and to make the upper edge of the outlet sharp. The inclination of the lower leg, the sharp upper edge of the outlet and the submerged outlet facilitate the escape of the air entrained by the water. The inclination causes the contained air to spread as a flattened layer against the roof of the lower leg, and in this position with a sharp outlet edge it is more easily removed by the escaping water carrying off the lower part until it is all drawn out. The submerged outlet has not been always used, but it probably prevents the entrance of air from the outside.

The great advantage of a siphon spillway over the ordinary overflow spillway is the close regulation produced by a siphon spillway not obtainable with an overflow spillway, unless the overflow crest is of such large length that its cost may be much higher than that of a siphon spillway. Topographic conditions will also determine the selection between the two types of spillways.

Gibswil Siphon, Switzerland (Fig. 70).—This siphon, which is rather unusual in the form of the inlet, is made of ¼-inch riveted steel pipe, tapering in diameter from 31.5 inches at the upper end to 23.6 inches at the outlet to prevent the water column from separating under the head of 52.48 feet. The inlet to the siphon is formed by the upper end of the pipe cut on a horizontal plane, covered by a reinforced concrete hood, with its lower edges extending 3.28 feet below the normal water surface. The air inlets consist of long narrow slots in three sides of the hood, placed at normal water level. Measurements of the flow gave a discharge of 98.9 second-feet, with some of the air open-

ings not fully closed; this gives a coefficient of discharge in the formula $Q = CA\sqrt{2gH}$ of 0.57. With the air openings fully closed, the maximum assumed discharge was 123.6 second-feet, giving a coefficient of 0.70.

Another type of siphon, built in Switzerland, is illustrated by the Seon spillway siphon (Fig. 71).

Siphonic Spillway of Tennessee Power Co. (Fig. 72 and Plate VIII, Figs. C and D).—The spillway consists of eight siphon units; four of them have operating head of 27.2 feet, and the other four 19.2 feet. In each unit the inlet is well submerged, with its upper edge 5½ feet under the water surface. The inlet is 3½ feet high and 6 feet wide, protected by ⅜-inch vertical screen bars, 4 inches center to center. The cross section tapers gradually to a height of 1 foot and a width of 8 feet at the throat. The vertical lower leg changes gradually from the throat cross section at its upper end to a width of 4 feet by a 2-foot height at the lower end, where it connects with the tapering enlarged outlet. The throat is protected by a casting. For each siphon, two air inlets, each 6 inches high by 18 inches wide, extend through the casting. The entrance to the air inlets is protected with ½-inch screen rods

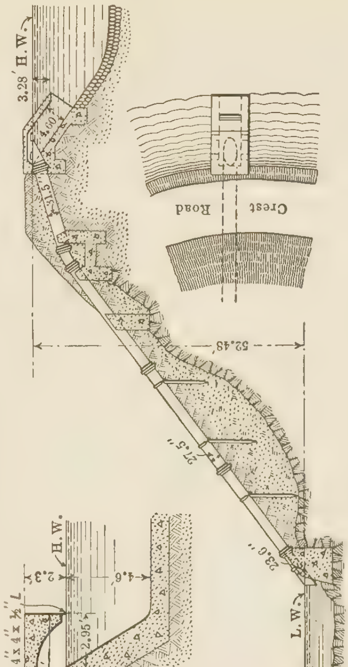


Fig. 70.—Elevation of high-head siphon at Gibswil. (*Eng. Rec.*, May 3, 1913.)

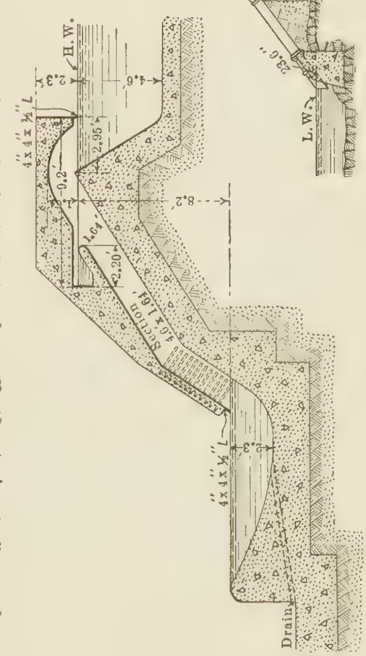


Fig. 71.—Spillway in use at Seon, Switzerland.

4½ inches apart. The upper edge of the openings is 3 inches above the overpour crest of the throat. The results of test give a coefficient of discharge, with the formula $Q = CA\sqrt{2gH}$ of 0.65.

A siphon spillway of similar design on the Glens Falls Feeder, New York State Barge Canal, acting under a head of about 5.50 feet, gave a coefficient of discharge of 0.62.

Siphon Spillway on Sun River Project, Montana (Fig. 73).—On the Pishkun reservoir supply canal and on the Sun River Slope Canal four siphon spillways have been projected. The type used is the same, varying only in dimensions. The smaller consist of two siphon units, designed for a combined capacity of 144 cubic feet per second, for an operating head measured from

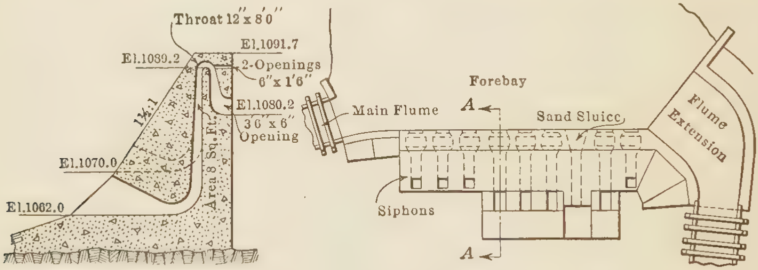


FIG. 72.—Cross-sectional elevation and plan of siphonic spillway for hydro-electric plant, Tennessee Power Co. (*Eng. Rec.*, May 16, 1914.)

the canal water surface to the center of the cross section of the outlet culvert of about 10.5 feet. The inlet to each siphon is placed with the upper edge 18 inches below the normal water level; the cross section is 3 feet square and tapers to a rectangular cross section at the throat, 3 feet wide and 2 feet high; the same rectangular cross section is used for the vertical downstream leg down to its lower end, where a sealing basin is formed with a tapering section which connects it with the outlet culvert. The air supply required to break the siphon action is obtained through a pipe system. The inlet to the pipe system is in the canal bank, 20 feet from the side of the siphon; it consists of an elbow screwed to the end of a 6-inch main wrought iron pipe and placed with the inlet facing down, at the normal water level. The inlet is enclosed in a small concrete box, the upstream face of which is opened and protected with ⅜-inch rods, 1 inch on centers. From this inlet box the main pipe extends along the downstream side of the siphon, near its crest, and branches with two 4-inch

wrought iron pipes which are connected to the crown of the throats of the two siphon units.

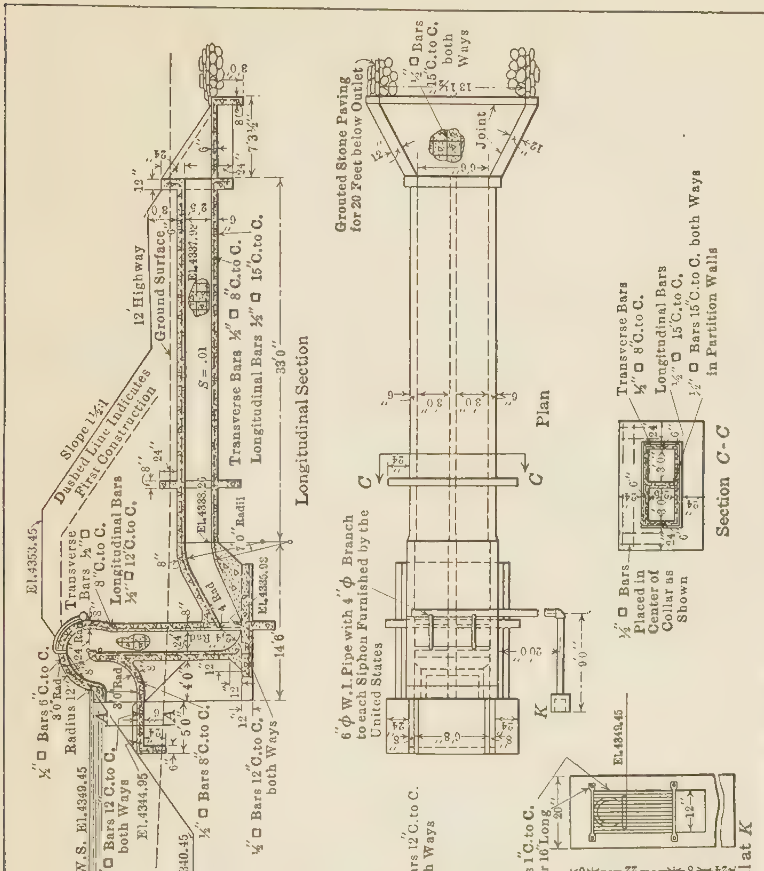


FIG. 73.—Siphon spillway. Sun River Project, Mont.

The hydraulic computations used in the design are as follows: The combined capacity of 144 second-feet gives an entrance

velocity of 8 feet per second; the entrance head is taken as ½ of the velocity head, or 0.5 feet. This leaves about 9 feet net head for the flow through the siphon, which for an assumed discharge coefficient of 0.50 gives a velocity of 12.0 feet per second. The culvert grade is 1 foot per 100, obtained by Kutter's formula, with a value of n of 0.012. The assumed coefficient of discharge is low as compared with the values previously given; this is especially so, as in these computations a separate allowance has been made for the loss of head in entrance.

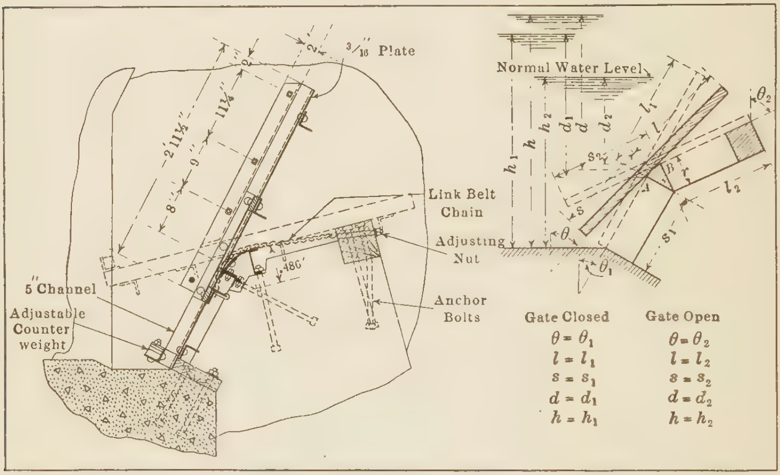


FIG. 74.—Automatic spillway gate for Dodson North Canal. Milk River Project, Mont.

Automatic Spillway Gates.—To maintain a constant water level in a canal, a number of ingenious types of automatic gates have been designed, which, when placed in a spillway structure or channel through the canal bank, will automatically open and close for variations in the water level of the canal above or below the desired level. These gates have been used to a very limited extent on a few projects, and for most of them their practicability is not demonstrated. The features which are most difficult to overcome are the leakage around the sides of the gates when closed, the comparatively high cost, and the uncertainty of operation.

The automatic spillway gate developed by I. B. Hosig and W. A. Perkins for use on canals of the Milk River project, Montana (Fig. 74) and the automatic gate used on a wasteway structure

of the Belle Fourche project, South Dakota (Fig. 75), are examples of some of the most promising types.

The automatic gate developed for the Milk River project (Fig. 74) consists of a rectangular gate supported on a curved or cylindrical horizontal surface, placed in such a position and of such radius that for variations in the water level above the desired water depth in the canal the gate in opening or closing revolves on a shifting axis, which maintains approximately the equality of moments of the water pressure above and below this point of support. The design is based on the following conditions of operations:

First.—When closed the gate is placed on an inclination to the vertical equal to θ_1 and is supported at a point A (the lower bearing edge of the cylindrical surface) located at distance s_1 from the lower edge of the gate, and is just balanced for a depth of water in the canal of h_1 equal to full supply depth (h_2) plus an excess usually equal to about 4 to 6 inches. The angle of inclination θ_1 and the height h_1 are first selected and determine the length of the gate $L \div l_1 + s_1$. The position of the point A is then determined by obtaining s_1 from the equation of equality of moments

$$M = \frac{s_1{}^2 d_1}{2} + \frac{2s_1{}^3}{6} \cos\,\theta_1 = \frac{(l_1 - s_1)^2}{2} d_1 - \frac{2(l_1 - s_1)^3}{6} \cos\,\theta_1$$

which reduces to

$$\frac{2}{3} \cos\,\theta_1(s_1{}^3 + l_1{}^3) = d_1(l_1{}^2 - s_1{}^2)$$

This equation is solved for s_1 by substituting the following values for d_1 and l_1:

$$d_1 = h_1 - s_1 \cos\,\theta_1 \text{ and } l_1 = L - s_1$$

Second.—When fully opened the gate obtains the inclination to the vertical equal to θ_2 and is supported at a point B (the upper bearing edge of the cylindrical surface) located at a distance s_2 from the lower edge of the gate, and is just balanced for a depth of water in the canal of h_2 equal to the even full supply depth. The value of s_2 is obtained from the equation of equality of moments in the same manner as for the gate closed, using the quantities s_2 d_2 l_2 h_2 and θ_2 in the place of s_1 d_1 l_1 h_1 and θ_1.

The requirement that A and B be on a curved surface determines the radius of the curve by the following relation between s_1 and s_2

$$s_2 = s_1 + 2\pi r \frac{\theta_2 - \theta_1}{360°}$$

or

$$r = \frac{s_2 - s_1}{2\pi} \frac{360°}{\theta_2 - \theta_1}$$

When designed to meet the above requirements, the gate will start to open when the water level in the canal raises a certain selected height $(h_2 - h_1)$ above the desired normal water level and will start to close from a wide open position when the water level drops down to the normal water level. For intermediate positions of the gate, the cylindrical bearing surface fulfills the requirements only approximately, such that when the gate is partly open the balancing head is slightly greater than the opening head h_1. The balancing head for any intermediate position may be determined analytically by substituting special values in the equations given above. Computations made on the Milk River project to determine the intermediate balancing heads for several types of gates, between the following limiting values:

h_1 between 4 and 6 feet.
h_2 from 4 to 6 inches less than h_1
θ_1 from 14° 02′ (¼ :1 slope) to 26° 34′ (½ :1)
θ_2 from 60° to 76° 47′

indicate that the maximum balancing head occurs when the gate is about half open and that it is somewhat less than 1 inch greater than h_1. For practical operation, the cylindrical bearing, although not the true shape to maintain an absolute constant water surface, is sufficiently accurate.

A wooden experimental gate, although rather roughly constructed, worked very successfully, maintaining a constant water level within 2 or 3 inches, for all positions of the gate.

The equations presented above do not consider the weight of the gate and the counterweight at the bottom of the gate, provided to hold the gate closed with the canal empty. In the final design the equation for equality of moments must be modified by entering the moment due to the counterweight (divided by 62.5) and the difference between the moments due to the weight of the gate above and below the pivot point (divided by 62.5).

The approved design of gates for a number of structures on the Dodson North Canal of the Milk River project is indicated by the cross-sectional view in Fig. 74. The spillway structures are to be built in widths that are multiples of 5 feet; and for each

structure the gate is formed of gate units 5 feet wide, bolted together. A combined spillway and escape structure equipped with these gates is shown in Fig. 77. One of the difficulties in the installation of the gates is the prevention of excessive leakage around the sides. The operation of these gates requires that there be no backwater pressure on the gates; for this, it is necessary to introduce sufficient fall below the outlet sill to carry away the water without its backing up against the downstream face of the gate.

The automatic gate for the wasteway structure on the Belle Fourche project (Fig. 75) is a cast-iron ribbed gate, on the downstream side of which are four arms. The two lower ones are bolted at their upper end to the bottom corners of the gate and at their other end form a curved surface with gear teeth which fit into a corresponding geared rack anchored in the side walls. The other two arms are diagonal braces, which with the lower arms transmit the total water pressure to the geared bearings.

The gate is placed between two side walls, and when closed is in a vertical position with the two side edges bearing on shelves formed in the side walls. The gate is so designed that the counter moment of its weight is greater than the overturning moment of the water as long as the water level in the canal is at or below the maximum full supply level; when the water level raises above this height the gate begins to open by revolving on the geared bearings and is fully opened when the raise in water level is 0.45 feet or more. The water discharged through the gate opening must be carried away without producing back pressure; this is obtained by the fall at the outlet and the steep slope of the waste channel.

ESCAPES

An escape structure controls the water taken out the canal through the canal bank and discharges it in an adjacent stream or close by natural channel (Plate VIII, Figs. A, B). The structure will usually consist of the regulator part constructed in the cut through the canal bank and the waste channel. The regulator part forms a number of gate openings and is similar in design to the headgates of the canal system and of main laterals. The gates must be of the undershot type if the structure is primarily used for scouring out deposited silt. The waste channel will usually be either a concrete lined or wooden chute, a pipe or box culvert,

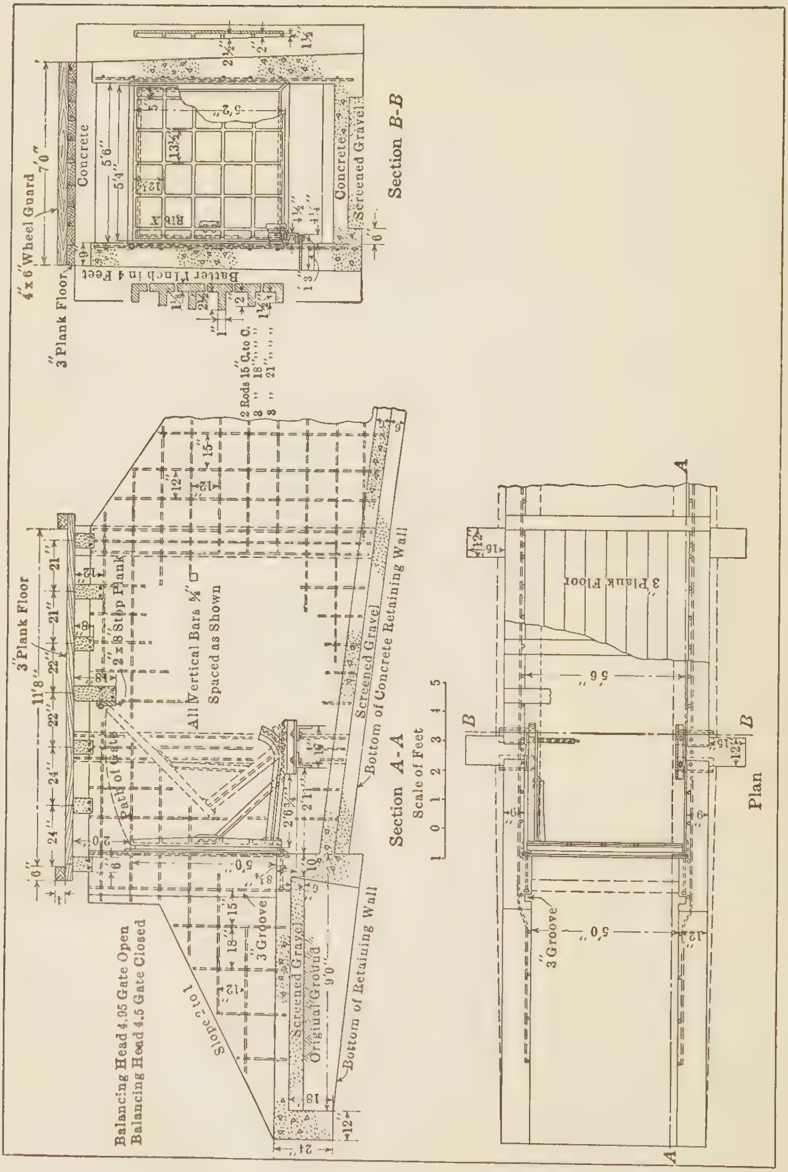


FIG. 75.—Automatic wasteway gate. Belle Fourche Project, S. D.

or an earth canal with a series of drops to take up the excess fall. When the escape is at about the same level and near the stream bed in which the water is discharged, no waste channel may be necessary and the small difference in elevation may be taken up with a single fall. The escape opening may at times be submerged, in which case the maximum water level in the stream channel, during the period that it may be necessary to use the escape must be determined as it may control the dimensions which must be given to the escape to obtain the desired capacity.

Necessity and Location.—An escape is, as previously stated, used for two main purposes:

First.—For the protection of the canal system.

Second.—To scour out the sediment deposited in the canal section above it, in which case it may be called a scouring escape. The structure can also be used to serve the purpose of a spillway.

The most important object is the protection of the canal system, and an escape designed for this purpose will usually meet at least to some extent the requirements of a scouring escape and of a spillway.

A scouring escape is designed specially to produce a high scouring velocity in the canal section above it. Where this action is confined to a comparatively short section of a canal whose cross section is enlarged to produce a lower velocity which will encourage the deposition of sediment, then the structure becomes what is commonly called a sand box or sand trap. The necessity of a scouring sluice may not be apparent when the system is first planned, and it will often be desirable not to provide one until after the necessity is well indicated. The necessity will depend on the volume of silt carried by the river water and the efficiency of the diversion works in preventing the entrance of sediment through the headgates. Where the amount of sediment entering the canal is liable to cause harmful deposits in the canal system, it will be desirable to confine the deposit in the upper part of the main canal. To obtain this the canal cross section for a length of canal, extending usually not more than 1 mile downstream from the headgates, will be made sufficiently large to give a normal velocity smaller than the velocity in the canal system below. The scouring escape will then be placed at the downstream end of this section of canal. No additional scouring escapes are usually necessary.

Where the water carries little or no sediment, no scouring escape

will be required. On the other hand, a number of escapes for the protection of the canal system will often be necessary. When they serve the purpose of spillway, their necessity and location will be determined from the considerations discussed for spillways. Important factors determining the need and location of spillways are the topographic conditions and character of construction. There are some irrigation systems where the diversion canal is constructed all in cut in comparatively level foothill land and the laterals are mostly balanced cut and fill, in flat valley land; so that there is little or no surface run-off from precipitation and few natural drainage channels. In such cases, especially if the main laterals are extended by a waste or tail channel to deliver into a natural drainage channel and their capacity down to the end made sufficient to carry any surplus unused water, the danger of canal breaks and resulting damages would not be serious and a few escapes at controlling points may be all that is necessary. Where such favorable conditions do not exist a larger number of escapes are necessary. An escape requires that it be situated within a comparatively short distance from a natural depression or channel of sufficient carrying capacity to carry the flow discharged through the escape. Such a channel usually exists near desirable sites for escapes. The investigation of its carrying capacity must not be overlooked, also the fact that it may be carrying natural drainage water at the same time. An escape is usually desirable or necessary for the following conditions and at the following points:

First.—When the bed of the diversion canal near the headworks is considerably lower than the flood plane of the river and the canal is so near the edge of the river bank that there is danger of the river flood flow cutting through or over the canal bank, or cutting a new channel around one end of the diversion weir into the canal; in either case the flood flow water, if allowed to continue down the canal, would do serious damage. For such conditions the escape must be placed at a point sufficiently far downstream from the headgate for the canal bed to have gained enough elevation or for the canal line to have swung sufficiently far away from the river bank to make the canal section downstream from this point safe from this danger.

Second.—When a length of canal is built along a steep side hill or along a steep bank of earth of comparatively soft material. In this location the flow of water through a break would

quickly enlarge the break for considerable distance upstream and if allowed to continue might wash out the side hill to such an extent that replacing the canal would be very difficult. An escape must be placed at the upstream end of the canal section, where it first approaches the steep bank, and when the length of canal in this dangerous location is large it may be desirable to place additional escapes at intervals of 2 to 4 miles, in order to be able to stop the flow through the break within a reasonable time.

Third.—At the upstream end of a section of canal which is constructed in treacherous material or of a long section of canal built in fill.

Fourth.—At the upstream end of important flumes or siphons, in order to prevent serious damage resulting from a leak around the inlet or outlet of the structure, or from obstruction in the siphon or break in the flume.

Fifth.—Just below points where large volumes of drainage water discharge or drain into a canal.

Sixth.—At the lower end of the diversion canal within a short distance from the division of the flow into the main laterals.

Capacity of Escape.—An escape used for the protection of the canal system below it must be designed for a carrying capacity equal to the maximum flow which may be carried in the canal. This maximum flow will be equal to the normal capacity of the canal, plus any surplus which may enter the canal.

A scouring escape is usually located within a comparatively short distance from the canal headgates and the canal in between made larger than the normal required capacity, in order that it may carry a surplus supply which can be wasted through the escape during the period of maximum stream flow when the water carries the most silt. In this case the capacity could be made only sufficient to dispose of this surplus supply; but, in order to be able to increase the scouring action and to use the escape also for the protection of the canal system, it will usually be desirable to make its capacity equal to the full maximum flow in the canal.

Action and Design of an Escape.—An escape structure may be combined with a checkgate, or may be constructed separately with no means, within a short distance below it, to check the flow. When combined with a checkgate, the water level in the canal may be held up to the full depth and the escape gates regulated to discharge the full canal supply, in which case the scouring

effect will be confined to a short distance near the headgates, as there will be no drop down surface curve.

When not combined with a checkgate, the structure may be designed to dispose of either part of or the whole canal supply. To be able to discharge the whole supply, which usually will be desirable, the floor of the canal in front of the gates and the gate sill must be placed at a depth which will give the required discharge capacity with no water continuing down the canal. The depressed canal floor may slope up at both ends to meet the canal bed, or may terminate with two end walls and with the side wall in which the escape openings are made form a short basin directly in front of the gates. To discharge the entire flow the water surface in this basin, with the escape gates fully opened, must not be higher than the bed of the canal. The flow from the canal into the basin will then be similar to the flow over the edge of a canal fall or drop, in which the edge over which the water pours is level with the bed of the canal. The depth of water at the edge of the fall may be obtained by the weir formula

$$Q = Cld^{3/2}$$

where l = length of overpour.

d = depth of water at edge of overpour.

C = a coefficient which the formulæ of Bellassis and Bazin would indicate to be from 4.75 to 5.00.

The flow from the basin through the escape gate openings may be obtained from the usual formula for flow through sluice gates, and where the lower edge of the gate when fully opened is above the water surface, the flow may be computed from the weir formula given above, modified where necessary for submergence.

The scouring effect will depend on the extent of drop down produced in the depth of water. To obtain a high scouring effect in the upstream canal section, the length of overpour into the basin may be made large so as to give a small value to the depth of overpour water.

The action of an escape, when disposing either of a part of the canal flow or the entire flow, may be illustrated by the example previously given for which the drop-down curve was worked out. The example will apply to either of the following cases:

First.—Assume that the escape disposes only of the surplus water, which is the difference between the flood-flow supply of 450 second-feet and the normal supply of 300 second-feet, and

that the normal flow continues downstream with a depth of water in the canal of 5 feet.

Second.—Assume that the entire supply of 450 second-feet is carried through the escape and that the length of overpour at the upstream edge of the depressed basin is made to give a depth of overpour of 5 feet. If the coefficient in the weir formulæ is taken as $C = 4.75$, then the length l must be $l = \frac{450}{4.75 \times 5^{3/2}} = 8.5$ feet, which is less than the bottom width of the canal and could only be obtained by contracting the width of the canal.

In both cases the scouring action on the canal section upstream is the same, and is due to the increased velocity produced by the drawdown which decreases the maximum full supply depth from 6.25 feet to the 5.00-ft. depth at the escape. The tabulated results show that, with a normal capacity of 300 second-feet, the normal velocity in the canal is 3 feet per second, and that when the flow is increased to 450 second-feet and the escape gates closed, the velocity is 3.39 feet per second. With this flow in the canal and the depth of water at the escape reduced to 5 feet, by operating the escape gates, there is a gradual increase in velocity from 3.39 feet per second, beginning 12,500 feet upstream from the escape to a maximum of 4.5 feet per second at the escape. When the canal section between the escape and the headgates is short, the increase for the whole length will be considerable.

The scouring effect of this increase in velocity is best explained by referring again to the principles of transportation of silt. According to Kennedy's theory, the transporting power of silt or the percentage of silt transported by water is proportional to $V^{5/2}$. In the example referred to, assume that the river water enters the canal with a greater percentage of silt than the canal water can carry at its normal velocity of 3 feet per second, and that to scour out the deposited sediment a larger volume is turned into the canal with the escape gates opened; this increases the velocity at the escape to 4.50 feet per second, and gives a corresponding silt transporting power of $\left(\frac{4.50}{3.00}\right)^{5/2} = 2.75$ times greater than with the velocity at which the deposit occurred. But at the upstream end of the reach affected by the drawdown, the transporting power is only $\left(\frac{3.39}{3.00}\right)^{5/2} = 1.36$ times greater.

The scouring effect will be further increased by the steeper grade produced by the larger deposit of silt toward the upper end of the reach affected by the drawdown.

When the escape is to be used primarily as a scouring escape, a high scouring velocity may be produced by designing the escape so as to obtain a large drawdown; this is regulated by the depth of water at the escape, which may be made small by increasing the length of the upstream edge of the depressed basin. On the other hand, a high scouring velocity may not confine its action to removing the silt, and if the canal is built in soft material may produce harmful erosion. Where it is necessary to protect the canal against this excessive erosion and at the same time not destroy the efficiency of the escape in silt removal, the canal section affected by the drawdown may be protected with a lining of concrete. The scouring effect can best be regulated and will produce better results if the escape is located within a comparatively short distance from the canal headgates, probably not over 1 to ½ mile and preferably less. The practice and experience in India indicates that to scour out silt deposits it is best to turn into the canal as large a flow as possible when the river water contains little silt. The use of fairly clear water is desirable, for with the little silt it contains it is able to pick up and transport deposited silt, while if the water is already well charged with silt, it cannot pick up much additional silt. For this reason intermittent flushes with fairly clear water have given the best results. To increase the scouring efficiency when the escape is at considerable distance from the headgates, the grade of the upper reach of the canal may be made steeper, so that a higher velocity will be produced. This is desirable because experience has shown that silt once picked up will be moved by a lower velocity.

When the escape must not produce a scouring effect, it may be objectionable to increase the velocity above its normal value, in which case the drawdown at the escape must be prevented by either using a check gate just below the escape, or by contracting the canal cross section just upstream from the escape. In order that the contraction will operate equally for varying discharges in the canal, the contraction may be made with a notch opening, the properties of which are discussed in notch drops.

Examples of Escape.—Fig. 76 shows a special type of escape, designed for the Dodson Canal of the Milk River project. It combines the action of an escape and a sluiceway. The canal is

designed for 500 second-feet capacity and may be ultimately enlarged to 900 second-feet. The structure consists of a culvert chute, placed through the canal bank, with its inlet connected by means of flaring wings to a depressed basin lined with grouted paving, and its outlet discharging into a concrete-lined stilling basin. The culvert is formed of two compartments, each 4 feet high by 4 feet 9 inches wide, down to a point 36 feet from the inlet, from which point it widens to 10 feet at the outlet. The inlet is regulated with straight lift gates and provision has been made for the use of flashboards in grooves formed in front of the gate

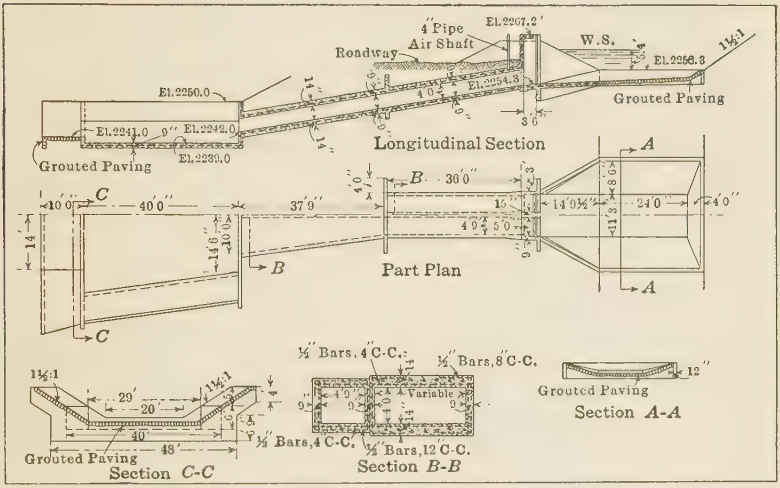


FIG. 76.—Escape on Dodson Canal. Milk River Project, Mont.

openings. A 4-inch pipe air-shaft connected to the upper end of each compartment, about 4 feet downstream from the gate opening, supplies air which makes the flow through the gate opening the same as a free discharge into air and equal to $Q = 0.80 a\sqrt{2gh}$ where h is depth of water on the center of the opening, a the area of gate opening, and 0.80 the coefficient of discharge assumed in this design. The flow down the culvert is accelerated, so that the culvert will run only partly full and is essentially a covered chute. The only apparent reason for widening the lower part is to spread the flow and decrease its depth before it discharges into the stilling basin. Provision is made to hold the water surface in the canal to full supply level by means of a

check gate below, for which condition the escape capacity is 558 second-feet, or 58 second-feet greater than the canal flow.

Fig. 77 shows an interesting design of a combined escape and automatic spillway for the Dodson North Canal of the Milk River project, Montana. The structure is located opposite a storm water overpour inlet, with a depressed basin lined with rubble concrete paving in between, formed in the bed of the canal in front of the escape gates. The waterway of the struc-

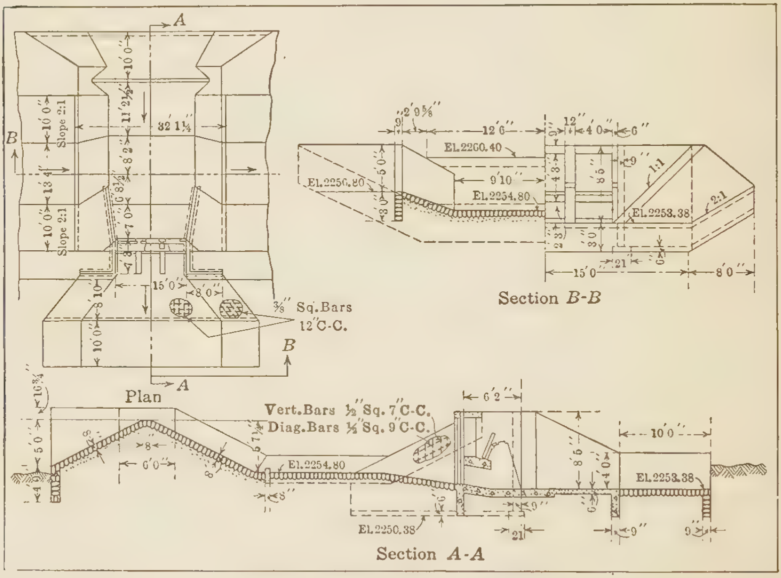


FIG. 77.—Escape and automatic spillway on Dodson North Canal, Mont.

ture between side walls is divided by a horizontal reinforced concrete slab and two intermediate division walls, supporting the slab, into three lower gate openings between the slab and floor and an overpour channel above the slab regulated with three automatic spillway gates of the type previously described (page 179). The three lower openings are controlled with straight lift gates. The downstream parts of the division walls or intermediate piers are extended above the slab and shaped to form the bearing, to which are bolted the cylindrical surfaces, about which the automatic gates revolve. Similarly shaped projecting surfaces or shelves are formed in the side walls for the end bearings. The

three lower gates are the escape gates and with the depressed basin combine also the action of scouring escapes. A number of structures of this type have been designed for installation on this project.

Other forms of escapes, in which the primary function is that of a sand box or scouring escapes, are illustrated by the examples presented in the discussion of sand box and by the inlet structure used on the siphon of the Kamloops Fruitlands Irrigation system (Vol. II, p. 345). Escape structures also form part of some of the examples presented in the discussion of surface inlet drainage crossings.

REFERENCES FOR CHAPTER IV

Murphy, D. W.—A Reinforced Concrete Spillway with Concentrated Crest Length—p. 278 Engineering News—Sept. 9, 1909.

The Siphon Lock on the Barge Canal at Oswego—p. 122 Engineering Record—July 30, 1910.

Reinforced Concrete Wasteways, Colorado—p. 124 Engineering Record —July 30, 1910.

Some Details of the Siphon Lock at Oswego, New York, Barge Canal—p. 114, Engineering Contracting—August 10, 1910; p. 530 Engineering News—Nov. 17, 1910.

Lauchli, E.—An Automatic Sluice Gate to Insure Constant Water Level, Bavaria—p. 711 Engineering News—Dec. 29, 1910.

Siphon Spillways, New York State Barge Canal—265 Engineering Contracting—Sept. 28, 1910.

Siphon Spillways on the Champlain Canal—p. 398 Engineering Record—Oct. 8, 1910; p. 398 Engineering News—Oct. 13, 1910.

Ludin, A.—Siphon Spillways, New York State Barge Canal—p. 467 Engineering News—Apr. 20, 1911.

An Automatic Sluice Gate for Spillway Crests—p. 602 Engineering News —Nov. 16, 1911; Engineering News—Dec. 29, 1910.

Hillberg, A. G.—Spillways of the Syphonic Type for Regulation of Storage Reservoirs and Forebays—p. 488 Engineering Record—May 3, 1913; also editorial, p. 481.

Adams, C. F.—Siphon Spillways—p. 54 Journal of Electricity, Power & Gas—July 12, 1913.

Siphon Spillway; System of Mt. Whitney Power & Electric Co.—p. 582 Journal Electricity, Power & Gas—Dec. 27, 1913.

Stickney, G. F.—Siphon Spillways on the Glens Falls Feeder, New York State Barge Canal—p. 338 Engineering News—Feb. 12, 1914.

Creager, W. P.—Syphonic Spillway for a Hydroelectric Plant, Tennessee Power Co.—p. 567 Engineering Record—May 16, 1914.

CHAPTER V

SAND GATES—SAND BOXES

These are structures designed to collect and remove from the water the sand or silt which is liable to be deposited in the canals, flumes or siphons of the system. The points along the canal where such structures may be necessary are:

First.—A short distance below the headworks to stop, as much as possible, the sediment from going farther downstream.

Second.—At the head of siphons to prevent at least the coarser material from entering the conduit of the siphon.

Third.—At favorable points along the canal where a drainage channel is crossed or is available for the discharge of the water and material sluiced out.

A sand-gate or sand-box structure is frequently designed to serve also as a wasteway or escape, and an escape specially designed to produce a high scouring velocity in the section of canal upstream and adjacent to the escape gates usually fulfills the purpose of a sand-gate structure. Such structures are illustrated by the inlet to the 4-foot siphon of the Kamloops Fruitlands system, British Columbia (Vol. II, Chapter X), and by the examples presented in the discussion of escapes. The principles of design of scouring escapes and the study of the scouring velocities produced by the drawdown created when opening the escape gates, presented in Chapter IV, will in general apply to the design of sand-gate structures.

Necessity.—The necessity of structures designed for the special purpose of removing sand or silt will depend on the extent and character of the sediment and on the means which have been taken in designing and operating the headworks to prevent the entrance of silt in the canal system and the extent to which the velocities in the canal system can prevent silt deposition.

The volume of sediment carried by irrigation waters and its distribution in the water has been considered in the general discussion of silt problems, Vol. II, Chapter IV. The finer silt carried in suspension by the water was shown to be comparatively

uniformly distributed in the water and is not always found in greater quantities toward the bottom of the canal. The coarser sediment of sand or gravel is not held in suspension, except with very high velocities, but is rolled and pushed along the bottom of the canal. The finer sediment may have considerable value as a fertilizer, in which case it is desirable to hold it in suspension in the water until it is delivered on the field. The extent to which this may be obtained will depend on the regulation of the velocities used in the canals, in accordance with the principles of design of canal cross sections for minimum silt deposit. But changes in velocities during the operation of the system, the obstruction produced by the growth of weeds, and other causes will tend to produce silt deposits. The coarser material entering the canal system may be controlled to some extent by the proper design of the headworks. In addition there is some coarser material produced from erosion of the canal bed and from the washing in of soil by the run-off from higher lands discharging into the canal.

Types of Structures.—There are two general types of structures. The first one is better adapted to waters transporting sand or coarse material; it consists of a sand trap which collects the sand and gravel as it moves along the bottom of the canal and discharges it either continuously or intermittently into a waste channel. The second type is formed by a settling basin whose cross section is made sufficiently larger than that of the canal to reduce the velocity of the water to a proportionate amount and cause a deposit of a portion of the material which is then washed out through sluicegates into a waste channel. The two types are often combined in one structure.

The first type, which may be called a sand trap or sand catcher, will usually consist of grooves, sand ducts, or small cross channels formed below the bed of the canal to catch the coarse material as it rolls along the bottom and of gates regulating the flow out of these channels through the canal bank into the waste channel. The flow through the gates must be regulated so as to sluice out the material collecting in the cross channels. During the period that the amount of material carried by the water is greatest, the gates may have to be left fully opened and the flow through them continuous. This period is generally during the flood flows, when the waste of water through the gates may not have to be considered; it may otherwise be desirable to operate the gates inter-

mittently. In the design of the sand trap the amount of water which may be wasted must be considered, and if operated continuously the canal capacity upstream must be made larger than the capacity downstream by the amount to be wasted. The size of the gate openings and of the cross channels depends on the maximum amount of water which is to be wasted. This type of sand box is illustrated by the structures used on the Amity Canal in Colorado (Fig. 78), on the Leasburg Canal, Rio Grande project, New Mexico (Fig. 79), on the Naches Power Co. Canal in Washington (Fig. 80).

The second type of structure, which may be called a settling sand basin, consists essentially of an enlarged basin in which the velocity of the water is decreased to cause the deposition of the transported material. The extent to which the material is deposited will depend on the reduced velocity and the length of time the water is in the basin, which will vary with the length of the basin. The design of this type of structure involves a consideration not only of the size of the basin when empty of deposited material, but also of the cross-sectional area of the basin when reduced by the deposit. The velocity must be smaller than the low velocities obtained in the canal system below; and as the basin is usually short as compared with the length of canals or siphon in which these low velocities are obtained, it is necessary that the velocity in the sand box be reduced to only a fraction of these low velocities. In selecting this minimum velocity, the velocities in the canals or siphons which must be considered are not those obtained when the canal system is operated at full capacity, but rather those corresponding to the minimum capacities at which the canals will be operated during the period which is liable to produce maximum silt deposits. The extent to which silt will be deposited depends also, as stated above, on the length of the basin, but the length which it is desirable to use must be comparatively short. This is due to the following conditions: The material transported by the water is deposited gradually from the coarser material at the upper end of the basin to the finer material at the lower end, as the velocity decreases gradually below the transporting velocity of the different sizes of particles. The deposited material has a tendency to compact and the cohesion between particles increases, so that the velocity which is required to erode and sluice out the material must be considerably higher than the transporting velocity of

the heaviest material and greater than the safe velocities against erosion previously given in another chapter. It is difficult to create these higher erosive velocities, for the effect of sluicegates, even if their capacity is large, is relatively small except for a short section of canal near them.

The erosive velocity which will be created in opening the sluicegates will depend largely on the area of the openings and the depth at which the openings are placed and is discussed in connection with scouring escapes (Chapter IV, pages 186 to 188). The gate openings are placed at the bottom of the settling basin. For effective sluicing the area of openings and the depth of the basin must be designed to discharge at least the full capacity of the canal with the water level at the gates lowered sufficiently toward the bottom of the canal to produce a drawdown in the surface water level, which will increase the slope of the water surface with a corresponding increase in the velocity for a considerable distance upstream. Where the structure is within a short distance from the headworks, the drawdown will permit forcing through the canal and sluicegates a discharge greater than the normal full capacity of the canal with gates shut, and thus increase the scouring velocity. An important consideration in the operation of the sluicegates is the fact previously discussed—that the power of water to pick up and transport additional silt will depend on the amount of silt which it already carries in suspension and that the practice and experience in India indicates the scouring effect to be greatest with clear water. Under the most favorable conditions it is difficult to sluice out large settling basins, especially by intermittent operation, which allows the deposited material to become compacted to a certain extent. The general practice has been to use short settling basins or boxes, in some cases using a combination settling box and sand trap in which the sluicing operation may be continuous to prevent the compacting of the deposit.

The settling basin may be of the following three forms:

1. The settling basin is an enlarged section of canal of considerable length (several hundred feet) with its bed depressed below the grade line of the normal canal section or with a raised sill or check gate across the canal at the lower end of the basin where it joins the normal cross section. This check gate can be used to advantage in increasing the depth of the basin. The sluicegates are placed at the lower end of the depressed basin or

upstream and adjacent to the raised sill or check gate, and may be built as a combined structure. The sills of the sluicegates will be placed level with the bed of the settling basin. The efficiency of this basin will be increased if a higher velocity is obtained by forcing an excess flow through the basin when the sluicegates are opened; this is usually only feasible where the enlarged section is within a short distance downstream from the headworks. This form of settling basin is illustrated by the structure built on the Umatilla project in Oregon (Fig. 81).

2. The settling basin is formed by depressing the floor of the canal and enlarging the cross section for a length of canal much shorter than in the form described above. The sides and bottom are generally lined with concrete. To obtain a gradual change in velocity and avoid undesirable eddies, the inlet and outlet are best formed with warped surfaces. The length of the basin should be at least six times the depth of water in the canal. The sluice gates are placed at the lowest point of the basin, and if the structure is to be used also as a wasteway they are made large enough to discharge the full supply capacity of the canal. In this form of basin the scouring effect is largely concentrated in the concrete lined basin. The structure is then a sluiceway and wasteway. The extent to which it will act as a settling sand box depends on the length and cross section of the enlarged basin.

3. The settling basin is a short section of canal or rectangular box divided by overflow cross walls into a number of compartments, with a sluice gate at the lowest point of each compartment. The water passes in succession from one compartment into the next, over the separating walls; the velocity being decreased by the greater cross-sectional area of the basin. The object of the overflow walls is to skim the upper part of water in the basin, to decrease the velocity more uniformly, and to give a better chance for the material to deposit. The length of the overflow walls may be increased by placing them on a sloping angle to the direction of flow instead of at right angles. The floor of each compartment is usually sloped downward toward the outlet gate to facilitate the sluicing out of the material. The outlet gate may be left open or partly open during the period that the water carries the most sediment, in which case it acts as a sand trap, or it may be operated only intermittently. The area of opening of sluicegates may be made large enough to act as a wasteway for the en-

tire flow, or it may be made only sufficiently large to sluice out the deposited material. This form of structure has been used especially where it is desired to remove the sand and silt before passing water into pipes used for irrigation, domestic water supply, or hydroelectric development. It is illustrated by the sand box of the Hemet Land & Water Co., Riverside, California (Fig. 83); and the sand box at the intake of the Denver Union Water Co., Colorado (Fig. 84).

SAND-TRAP TYPE OF STRUCTURE

Sand Trap and Escape Gates on the Amity Canal, Colorado (Fig. 78 and Plate IX, Fig. A).—This structure, built in 1905, shows one of the interesting, comparatively early uses of reinforced concrete for the construction of irrigation structures, and its special adaptation in this case to a type of wooden structure, known as the Land Sand Gate, used for many years in Colorado.

The capacity of the structure is 870 cubic feet per second. It consists of regulating or check gates across the main canal with waste and sand gates at right angles. Upstream from the regulating gates and in front of the waste gates there are two floors on different levels; the upper floor is at about the same level as that of the sill of the check gates and of the canal grade below the check gates. It covers part of the lower floor, placed on about the same level as the bed of the upstream canal, which is depressed below the normal downstream grade. Between these two floors are formed channels or sand ducts by curved ribs, which also support the upper floor or cover. Only part of the cover over the upper ends of the ducts is permanent; it is reinforced and is 2 inches thick; the downstream part is made of reinforced cement mortar planks (1 part of cement to 3 of sand), which may be removed in case of obstruction of the ducts. These planks are 8 inches wide, 1¼ inches thick, made in two lengths 6 feet 2½ inches and 10 feet 2½ inches, and reinforced with ¼-inch steel rods. The inlet end of each sand duct has an opening 9 inches high and about 3 feet 3 inches wide; the width of the sand duct is made smaller toward the outlet, but the cross-sectional area of the duct is kept constant by a gradual increase in height to a maximum of 18 inches at the outlet. The object of the two floors and the sand ducts is to catch and draw through these separate channels the material from the far side of the canal, to the same extent as that from the near side. With the maximum depth of water in the

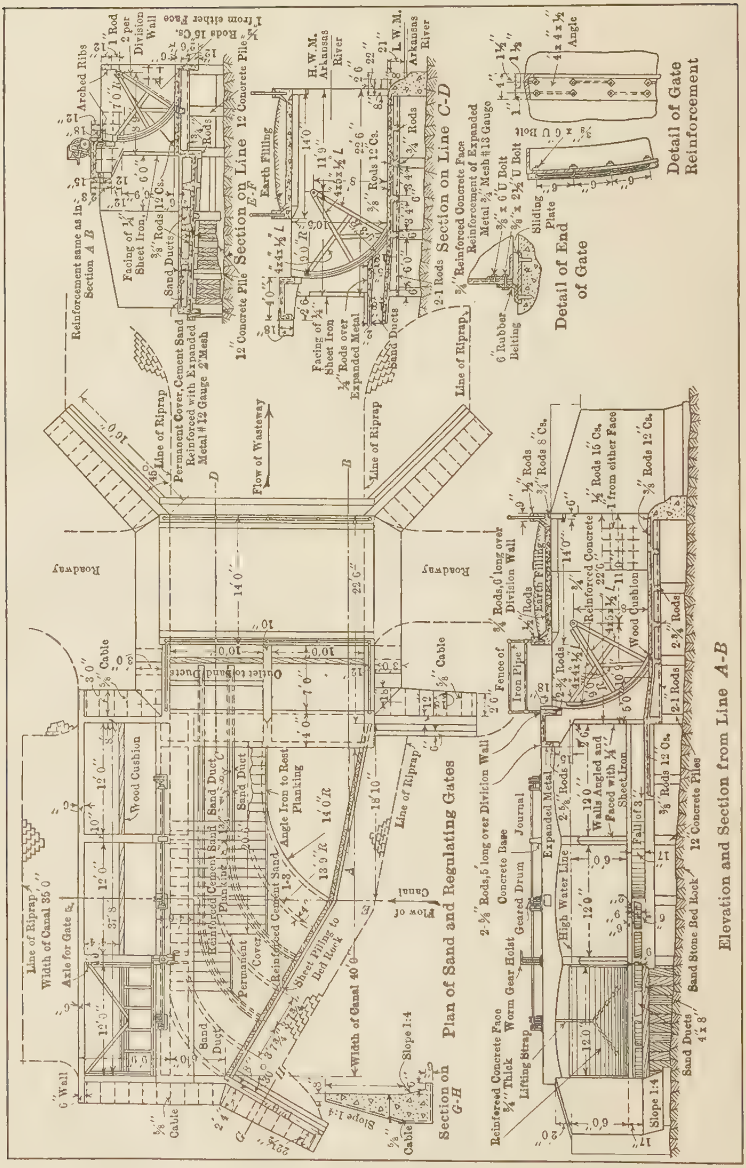


FIG. 78.—Sand escape and regulating gates on Amity Canal. Arkansas Valley Sugar Beet and Irrigated Land Co., Colo.

Fig. A.—Sand trap and waste gates on the Amity Canal. The Arkansas Valley Sugar Beet & Irrigated Land Co., Colo.

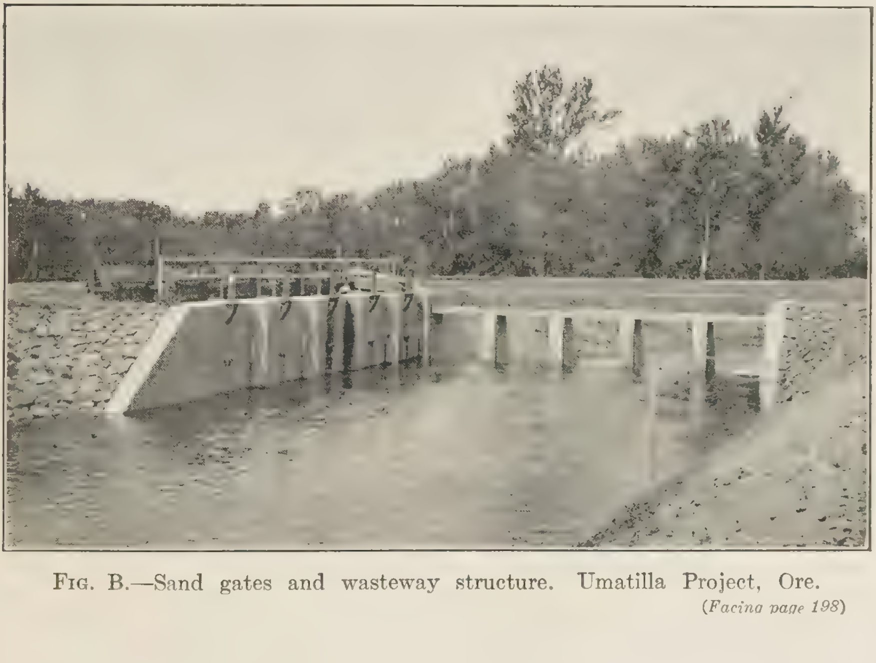


Fig. B.—Sand gates and wasteway structure. Umatilla Project, Ore.

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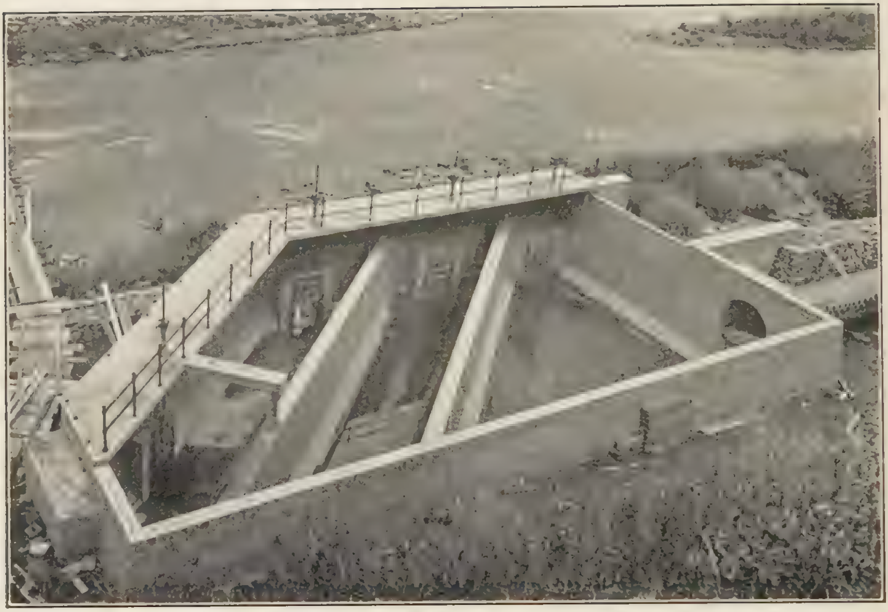

FIG. C.—Sand box on new water supply intake pipe line of Denver Union Water Co., Denver, Colo.

FIG. D.—Sand box on new water supply intake pipe line of Denver Union Water Co,. Denver, Colo.

canal, there is a head on the center of the inlet to the ducts of about 7 feet; with a free discharge at the outlet and considering the channel as a short tube, the velocity will be about 12.75 feet per second and the discharge of each duct 31 cubic feet per second. The sand ducts are opened continuously during the period that the water carries considerable sediment, which occurs when the river is in its high stage.

The regulating gates control the flow of water down the main canal. They consist of three openings, each 12 feet wide, separated by buttresses 10 inches wide, supporting the operating platform. Each opening is controlled by a radial or Taintor gate. The wasteway and sand gates consist of two openings, each 10 feet wide, controlled also by Taintor gates. The maximum depth of water against the regulating gates is 6 feet and against the wasteway gates 8 feet 5 inches. The gates are formed by a segment of a cylindrical shell fastened to a framework of curved ribs with braces and radial arms extending to and bearing on the pivoting points. The entire framework is built of angles. The face of the gates is a very thin shell of concrete, ¾ of an inch thick, reinforced with ¾-inch mesh expanded metal fastened to the steel angle ribs by means of V-bolts. To make the gates water-tight the bottom of the gate rests on a wood cushion, and to the sides of the gate rubber belting is fastened, which bears on the steel sliding plates bedded flush in the sides of the piers. These radial gates permit the use of wide gate openings, which is advantageous when, as in this case, the water in the canal is liable to carry ice, which with narrow openings is more likely to cause ice jams.

The foundation of this structure is formed of 12-inch concrete piles resting on sandstone, three under each buttress wall, two under the floor above the regulating gates, and one under the floor below the sand gates. These piles support reinforced concrete beams, on which is built the floor. To prevent undermining, sheet piling at the upstream end and concrete walls at the downstream end extend down to the sandstone stratum. The structure contains 230 cubic yards of reinforced concrete and 135 cubic yards of plain concrete, and required 1,151 cubic yards of excavation. The total cost was as follows:

Excavation, concrete and backfilling complete....	$4,160.08
Structural steel for radial gates..................	953.52
Hoisting devices for radial gates................	275.00
	$5,388.60

Sand-trap Sluiceway on Leasburg Canal, Rio Grande Project, New Mexico (Fig. 79).—This structure is of the same type as that described on the Amity Canal, Colorado. It is located 6,000 feet below the headworks on the main canal of the Rio Grande project, which diverts water from the Rio Grande River.

The structure consists of four channels or sand ducts, separated by ribs or partitions formed between the two floors; each is regulated by a gate placed between the inlet and outlet ends of the duct. The lower floor is near the level of the bed of the canal

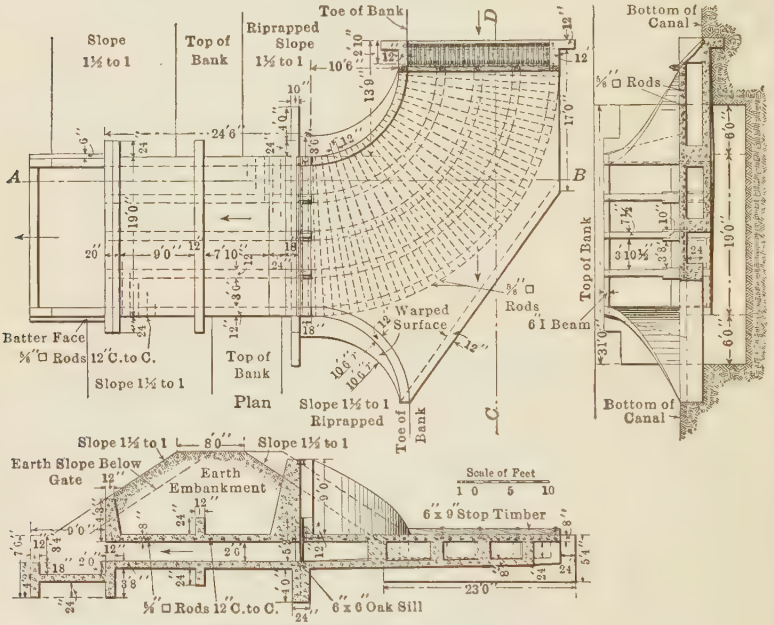

FIG. 79.—Sand trap sluiceway on Leasburg Canal. Rio Grande Project, N. M.

on the upstream side and the upper floor is about flush with the bed of the canal on the downstream side. The difference in elevation is about 3 feet.

The inlets to the sand ducts are placed across the bed of the canal to catch the coarser material carried near the bottom of the canal. The upstream parts of the sand ducts from the inlet to the gate are curved through an angle of 90° to deliver the water through the canal bank. The inlets to the sand ducts are protected against the entrance of large material, which might cause obstruction, by

grating bars placed on a slope of about 45° spaced 7 inches center to center with a clear width of 6 inches.

Sand Trap on Naches Power Canal, Washington (Fig. 80).—This structure is used on a power canal to catch and remove the heavier material carried by the water along the bottom of the canal. The canal is concrete-lined. The trap is formed in the floor of the canal by a gradual depression in the floor of 18 inches in a distance of 20 feet, which directs the material into four narrow rectangular openings, which are the inlets to four sand ducts.

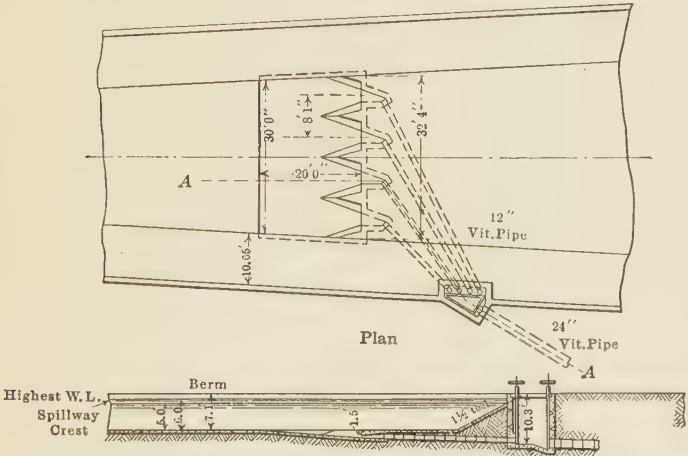

FIG. 80.—Sand trap. Naches Power Canal, Wash.

These openings are 4 inches high and 2 feet 6 inches wide; the top edge of these openings is 14 inches below canal grade. The sand ducts are made of 12-inch vitrified pipes, connected to the openings by inlet sections 4 feet long, changing in cross section from the dimensions given above for the opening at the inlet to 12 inches by 15 inches at the junction. The four 12-inch pipes are extended across the canal to discharge into a well or box, from which the water is let out through a gate into a 24-inch vitrified pipe. The outlet of each 12-inch pipe is regulated by a gate; the only apparent need for this is to confine the sluicing action to three or less of the sand ducts.

SETTLING BASIN TYPE OF STRUCTURE

Umatilla Sand Gates and Wasteway, Oregon (Fig. 81, and Plate IX, Fig. B).—The settling basin is formed by the canal

section 1,600 feet in length, extending from the headgates of the canal system to the regulating weir and gates across the main

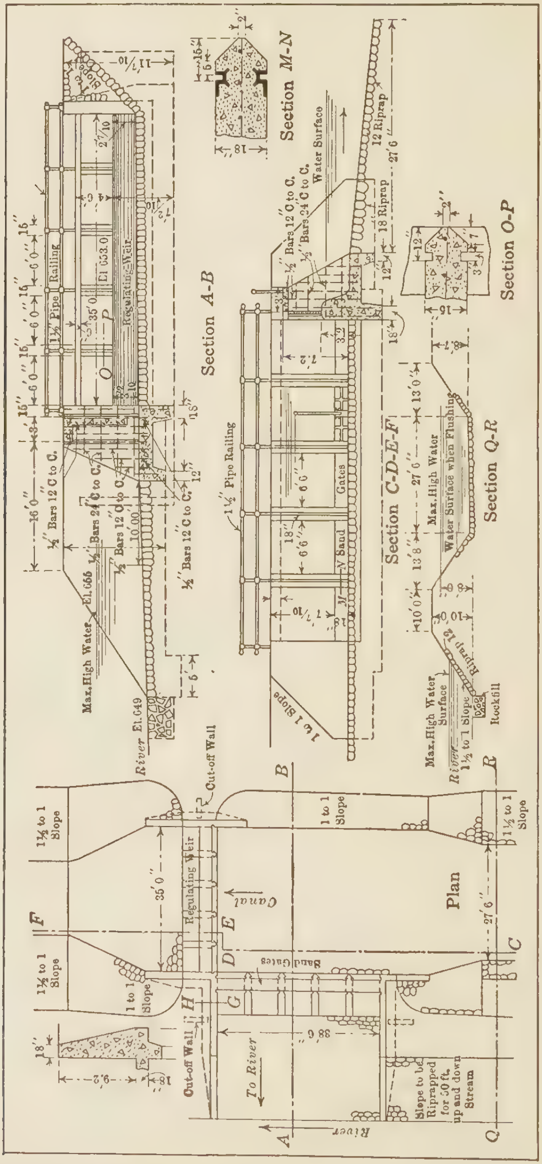

FIG. 81.—Sandgates wasteway and regulating weir. Umatilla Project, Ore.

canal, which forms one structure with the sand gates at right angles to it on the upstream side. The crest of the regulating

weir is about 3 feet higher than the bottom of the canal; the depth of the basin can be further increased by the use of flashboards. The cross section of the canal which forms the settling basin decreases gradually from a bottom width of 66 feet 3 inches at the headgates to a bottom width of 27 feet 6 inches at the sand gates. To discharge the normal full capacity of the canal of 300 cubic feet per second with the five sand gates fully opened, each 6 feet 6 inches wide and 18 inches high, there must be a difference in water levels between the upstream and downstream sides of the sand gates of about 1.25 feet. During low stages of the river this normal flow and a considerable excess can be discharged through the sand gates without the water level rising above the weir crest. During maximum flood stage of the river the water level in the settling basin may be raised above the flood height by the use of the flashboards. The cross section of the settling basin permits the carrying of a large excess of water, which can be wasted and used for flushing out the deposited material through the sluice gates. The entire bed and the sides up to about 2 feet in depth are protected against erosion of the natural earth surfaces by riprap. To facilitate sluicing, the bed of the canal near the sand gates has a cross slope of about 9 inches toward the gate openings.

The itemized cost of construction is tabulated below. Freezing weather and high water in the river increased the difficulty and cost of construction; seepage water would stand to a depth of 2 feet above the bottom of the excavation. The concrete used was mixed in the proportion of 1 cement : 2.5 sand : 5 gravel.

Construction Cost of Umatilla Sand Gates and Wasteway

	Actual quantity	Unit cost	Total cost
Excavation	219 cubic yards	$2.74	$601
Concrete (excluding cement)	268 cubic yards	6.83	1,830
Steel reinforcement	1,900 pounds	0.033	63
Cement	338 barrels	2.17	733
5 C. I. gates and guides	762 pounds	0.074	564
5 hoisting devices	3,425 pounds	0.14⅔	503
Setting iron and steel	12,945 pounds	0.004	43
Pipe railing	....	....	201
Riprap	318 cubic yards	5 86	1,863
Rock fill, river bank	209 cubic yards	1.24	259
			$6,660

Settling Basin, Sluiceway and Sand Gate on Lower Yellowstone Project, Montana, North Dakota (Fig. 82).—This structure con-

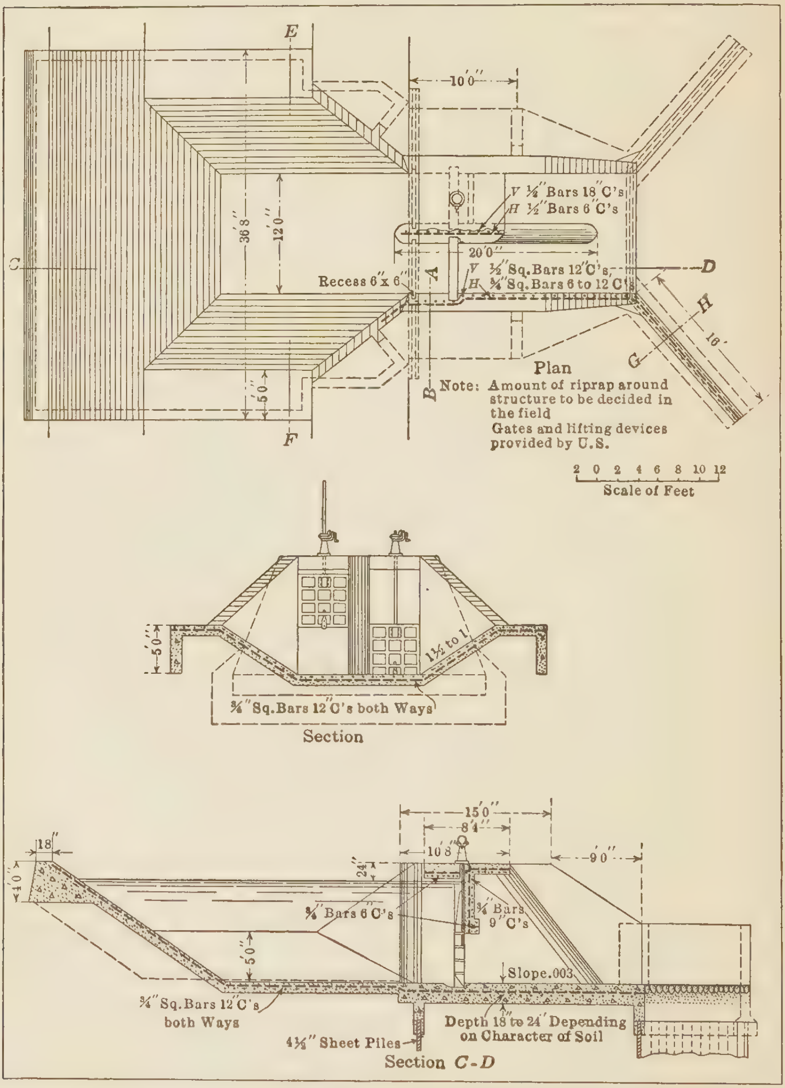

Fig. 82.—Standard settling basin sluiceway and gates. Lower Yellowstone Project, Mont.-N. D.

sists of a short settling basin, formed by a depression in the floor, and of sluicegates. The openings are below the normal bed of the

canal and are of sufficient size to discharge the entire flow of the canal, with the water level on the upstream side of the gate only a short distance above the top of the gate opening, leaving only a small depth of water in the canal downstream. The structure is then practically a wasteway as well as a sand box and sluiceway. On account of the short length of the basin formed by the depression only the coarser material rolled along the bottom of the canal would be caught or settled in the basin. With the gates fully opened, the drop in water level at the gates will extend for a considerable distance upstream and produce a higher velocity, which will produce erosion not only of the material deposited in the basin but also to some extent in the upstream canal, which if made sufficiently large to give a low velocity and cause settlement may be considered as part of the settling basin.

COMBINATION SETTLING SAND BOX AND SAND TRAP TYPE

Sand Box of the Hemet Land & Water Co. (Fig. 83).—This box is built at the junction of a concrete-lined canal and a wooden pipe. The box is divided by partition walls into three settling

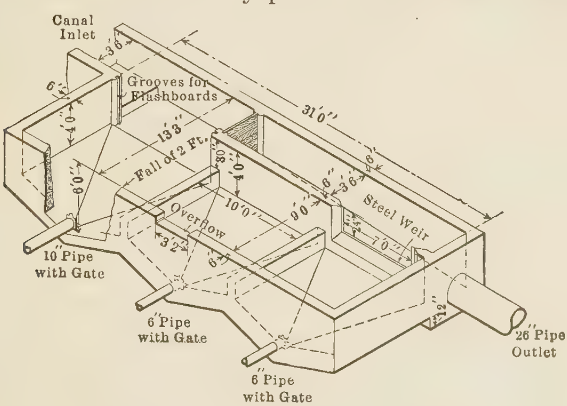

FIG. 83.—Sand-box. Hemet Land and Water Co., Calif.

basins. The water is carried to the box in the concrete-lined canal and is diverted into the first compartment by the insertion of flashboards across the canal. It passes in turn into the other compartments, flows over the measuring weir and enters the pipe. By removing the flashboards across the canal and closing the entrance to the settling basins, the water will pass into the pipe directly. This may be necessary in case of obstruction of the sluice openings and pipes.

Sand Box, Denver Union Water Co., Colorado (Fig. 84 and Plate IX, Figs. C and D).—This structure forms part of the works of the new water supply intake of the Denver Union Water Co. The works consist of the diversion dam and headworks on the South Platte River, a diversion tunnel 70 feet long, through the ledge below the intake chamber at the headworks, continued with 225 feet of 60-inch wooden stave pipe, incased in a 6-inch shell of reinforced concrete, down to the sand box.

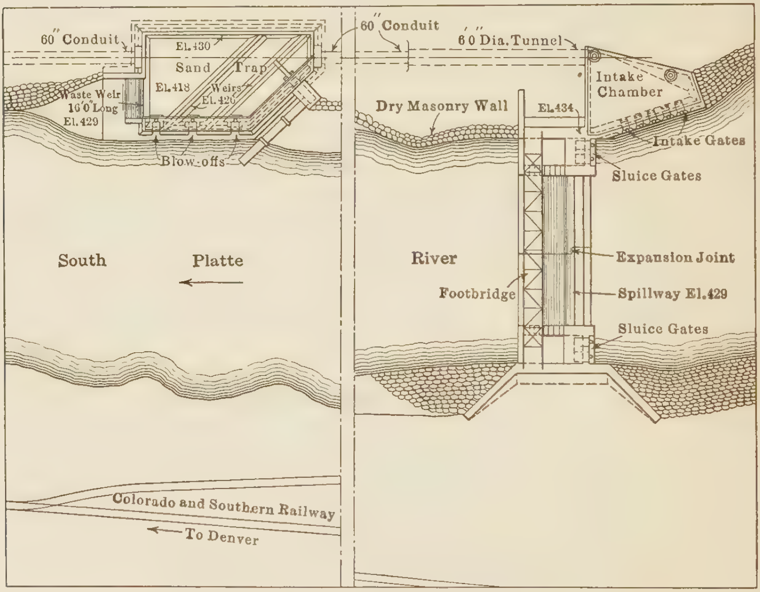

Fig. 84.—Plan of new head works for Denver Water Supply. (*Eng. Rec.*, Oct. 19, 1912.)

The water is taken out of the box and conveyed in a 60-inch wooden stave pipe. The sand box, built of reinforced concrete, is trapezoidal in plan, with outside dimensions about as follows: 68 feet on longest side, 43 feet 6 inches on shortest side, and 40 feet wide. The inside depth is 12 feet. The walls are 12 inches thick on top, and have a batter of 12 inches on the inside. The floor is 2 feet thick, reinforced top and bottom. The wall of the box near the river bank has a cut-off wall extending 6 feet to prevent backwash from the sluicing-out operation. The box is divided into three compartments by diagonal baffle or overpour division walls, placed at 45° to the longitudinal axis of the box. These walls are 16 inches wide on top, battered on both sides to

2 feet wide at the bottom; their crests are 4 feet below the top of the walls of the box and 3 feet below a spillway crest, 16 feet wide, formed in the downstream end of the box. When in operation, the flow is regulated at the headworks to give a small excess over the spillway, and thus maintain a constant head on the pipe line, with the water level about 3 feet above the crest of the overpour division wall. Under these conditions and with the rated capacity of about 82 cubic feet per second, the water cross-sectional area of the box gives a velocity of about 0.15 feet per second.

The sluicegates are cast-iron ribbed gates, 3 feet square. The first or upstream compartment has two sluicegate openings, one near the inlet to the box, discharging through a 3-foot cast-iron pipe into the river. The other three sluicegate openings in the wall adjacent to the river edge discharge directly into the river.

REFERENCES FOR CHAPTER V

Sand Gates in Canals—A Discussion in Annual of the American Society of Irrigation Engineers—1892–1893—Denver, Colo.

Thomas, D. G.—New Water Supply Intake at Denver—p. 424 Eng. Record—Oct. 19, 1912.

A Hydroelectric Power Plant in Chile—p. 356 Eng. Rec.—Sept. 28, 1912.

Distribution System in Pueblo Rocky Ford Irrigation Project—p. 603 Eng. Rec.—Nov. 18, 1911.

Pueblo Rocky Ford Irrigation Project—p. 141 Eng. Rec.—July 29, 1911.

Intake to Wood Stave Pipe Line, Willow River Irrigation Project—p. 505 Eng. Rec.—May 20, 1911.

Sand Trap, Glenwood Hydroelectric Plant, Central Colorado Power Co.—p. 808 Eng. Rec.—June 25, 1910.

Sand Box, the Pipe Line of the Cañon City Water-supply System—p. 622 Eng. Rec.—Dec. 4, 1909.

Sand trap, Bijou Irrigation District, Colorado—p. 9 Eng. News—Jan. 3, 1907.

Crafts, H. A.—A Sand Trap for Irrigation Ditches—p. 150 Eng. News—Aug. 11, 1906.

Sand Box, The Power Plants of the Edison Electric Co., Los Angeles—p. 213 Eng. Rec.—Feb. 25, 1905—p. 303 Eng. Rec.—Mar. 11, 1905.

Hall, Wm. H.—Sand Box, Santa Ana Canal of the Bear Valley Irrigation Co.—p. 61 Vol. XXXIII, Trans. Am. Soc. C. E. —1895.

CHAPTER VI

CROSSINGS WITH DRAINAGE CHANNELS

Object of and Types.—Crossings are required wherever it is necessary to construct a canal across a drainage channel. They will be most frequent on canals located across the drainage slopes, such as on diversion canals constructed along steep side hills and on main canals located along the foothills. The natural drainage channels will usually be well-marked depressions and may vary from small gullies and creeks to wide deep depressions or streams of considerable size. In flat country or along side hills of gentle and uniform slope the need for crossings may not be indicated by well-marked depressions but may nevertheless exist, in which case the topography of the land above the canal must be studied and a crossing located at the point most favorable for the collection of the run-off water. Drainage crossings will not usually be necessary on the main laterals of the distribution system, as these will generally be on the ridges. Drainage crossings are made by the following methods:

First.—By an intercepting artificial channel constructed uphill from the irrigation canal, to collect run-off water from minor channels and divert it to a main or collecting channel.

Second.—By passing the irrigation water over the drainage channel, usually by means of an elevated flume, and in some cases of long inverted siphons by carrying the lower part of the pipe line on piers or a bridge above the waterway of the drainage channel.

Third.—By passing the irrigation water under the drainage channel by means of an inverted siphon.

Fourth.—By passing the drainage water under the canal bed by means of a culvert or inverted siphon.

Fifth.—By passing the drainage water over the canal by means of a flume called an overchute.

Sixth.—By passing the drainage water into the canal and in some cases through the canal by a structure called a level crossing.

The selection of the method of crossing and type of structure

will depend on the relative elevation and positions of the canal and the drainage channel, the volumes of drainage and canal water, the uncertainties affecting flood flow, the effects on the safety and the operation of the system, economic considerations and other factors.

Intercepting Channel and Diversion Works.—This type of works is used only occasionally. In its simplest form it may be used to protect a canal on a side hill from the surface run-off, draining from the uphill slope, in which case it will consist of an intercepting ditch placed uphill from the canal and transverse to the slope, and discharging the collected water at intervals through a culvert or in an overchute across the canal. The only instance

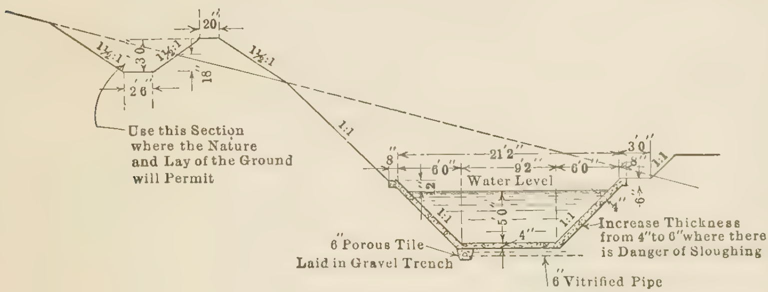

FIG. 85.—Concrete lined canal, with intercepting drainage ditch. Naches Power Co., Wash.

the writer knows of the use of this type of works is for the protection of concrete-lined canals on the following projects: The power canal of the Naches Power Co. in Washington, as shown in Fig. 85; the main diversion canal of the Tieton Canal in Washington, and that of the Kamloops Fruitlands Irrigation & Power Co. in British Columbia. On this latter system the intercepting ditch with frequent overchutes was found necessary along those sections of the main canal in sidehill land sloping on a comparatively gentle and uniform slope toward the canal, where considerable run-off in the spring months resulted from the rapid melting of the snow by the action of warm rains or early warm weather and winds. At a few such places where no provision had been made to intercept the run-off, a few concrete slabs of the uphill side slope lining were pushed in by the pressure of the water behind the lining, and in some places the canal was filled to overflow.

This type of works may also be feasible where two or more drainage channels close together can be connected by an inter-

cepting ditch or cross ditch on the uphill side of the canal to divert the flow from these channels into that of the main channels and use a single crossing. But in many cases, on account of the substantial diversion works required, especially for larger channels, separate crossings for each channel will usually be more economical.

Flumes to Carry Irrigation Water over the Drainage Channel.—This type of works is used when the bed of the irrigation canal is at a sufficient elevation above the bed of the drainage channel to give a waterway under the flume amply large enough to carry the maximum flood flow of the channel. Flumes are extensively used to cross drainage channels and depressions. In many cases the flood waterway will be only a comparatively small channel at the deeper part of the depression. Where there is any doubt regarding the adequacy of the flood waterway under the structure, such as with comparatively short flumes, elevated at small heights above the stream bed, careful consideration of maximum flood flows is necessary. Where a deep, wide, important stream is to be crossed, the flume may in many cases be carried across more economically on a truss bridge than on trestle. The same type of works and considerations will apply to the lower part of an inverted siphon pipe line, carried on trestle or on a truss bridge across the channel or stream.

The use and design of flumes has been discussed in Vol. II, Chapter IX.

Inverted Siphons to Carry Irrigation Water under Drainage Channels.—This type of structure is preferable for the following conditions:

First.—When the elevation of the canal bed is not at an elevation sufficiently above the bed of the drainage channel to permit the use of a flume crossing with the necessary flood area of waterway under the flume.

Second.—When the volume of flood waters in the drainage channel is greater than the volume of water carried by the canal.

On account of the uncertainties on which the estimates of the volume of flood waters must be based, it is the safest and the most desirable type of crossing where the conditions stated above exist. The inverted siphon may consist of a short conduit depressed below the bed of the drainage channel, under comparatively little hydrostatic pressure, with suitable inlet and outlet structure; or be a long pipe-line siphon, under considerable hydro-

static pressure, which crosses the channel of the drainage water at the lowest part of the depression. The use and design of inverted siphon pipe lines and the considerations involved in the

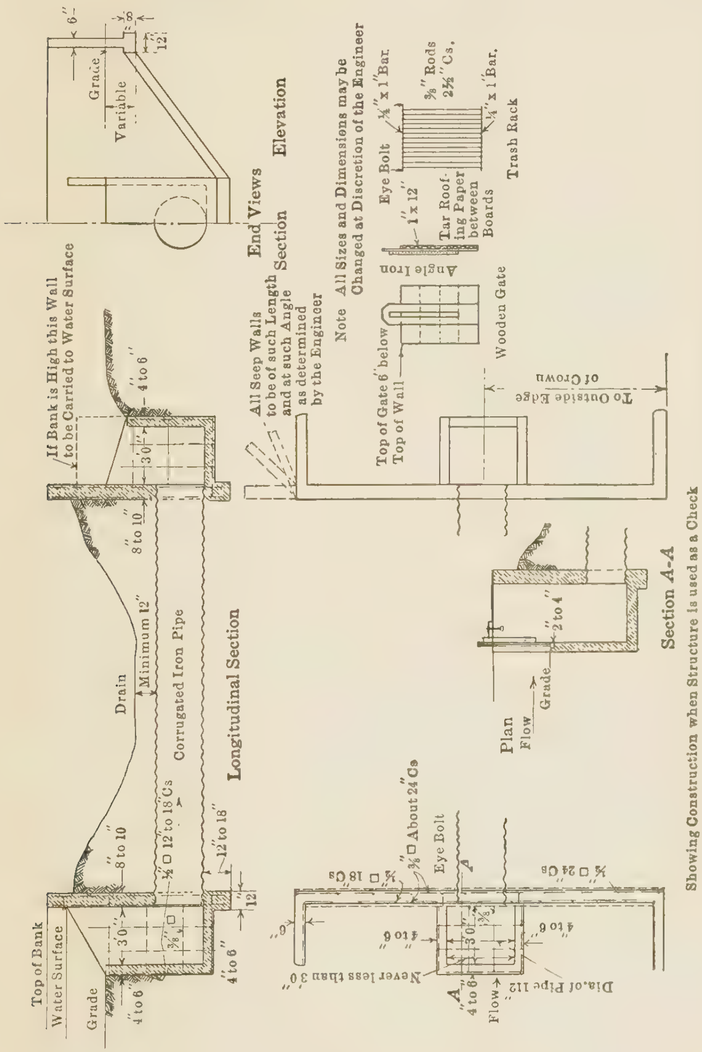

FIG. 86.—Inverted siphon culvert for drainage crossing. Sacramento Valley Irrigation Co., Calif.

selection between carrying the lower part of the pipe line under the bed of the drainage channel or elevated above the flood waterway for a stream of considerable size have been presented in Vol. II, Chapter X. Inverted siphons consisting of a short

conduit under low hydrostatic pressure are essentially the same as those used for roadway crossings, discussed and illustrated with examples in Chapter XI. Fig. 86 shows a simple type of inverted siphon corrugated pipe culvert used by the Sacramento Valley Irrigation Co., California, for crossing of laterals with shallow drains. The lateral flow is carried under the drain and the inlet structure may be modified for use as a check gate, with the addition of a wooden gate placed at the entrance. For check gate purposes an overpour flashboard gate would be preferable to the undershot gate.

Culvert or Short Inverted Siphons for Carrying Drainage Water under Irrigation Canals.—This type of structure is usually economically feasible only for drainage channels whose maximum flood flow is relatively small and not larger than the flow of the canal. As compared with an inverted siphon carrying the irrigation water under the drainage channel, the uncertainties of flood flows and the added safety obtained by not changing the flood flow channel will justify a greater cost for a structure taking the irrigation water, whose flow is known, under the drainage channel. The culvert form is used when the elevation of the drainage channel is sufficiently below the bed of the canal to obtain the required waterway cross-sectional area without depressing the conduit below the bed of the channel. The inverted siphon form differs from the culvert form in the position of this conduit. The design of the culvert or inverted siphon is essentially the same as that used for roadway crossings (Chapter XI). The conduit must be placed sufficiently low to have the entire cross-sectional area at least below the usual flood flow water level in the drainage channel or depression and preferably lower than the average level of the adjacent land; this is important, for if the conduit is placed too high it will cause backing up of the water on the upstream side, which may result in the flooding of land above. The hydraulic computations involve a careful study of all available data to estimate the maximum flood flows which may be expected. The determination of the required cross-sectional area may be made on the assumption that for extraordinary floods the water level at the inlet will be raised within a short distance of the top of the canal bank. Protection of the outlet is necessary where a high outlet velocity will be obtained. The outlet velocity can be reduced to a safe value if the conduit is placed sufficiently low to form the outlet as a basin depressed below the natural ground sur-

face with tapering wings. With a properly designed outlet the velocity through the conduit of the culvert may be as high as 15 to 20 feet per second. The inlet must be designed to give a strong connection with the canal bank. The liability of obstruction by large débris carried by flood flows must be considered in determining the size of the openings of the conduit, and may prohibit this type of structure.

A simple pipe culvert in which the pipe entrance is depressed below the natural ground surface is illustrated by the type of structure used to carry small drainage flow under the Dodson

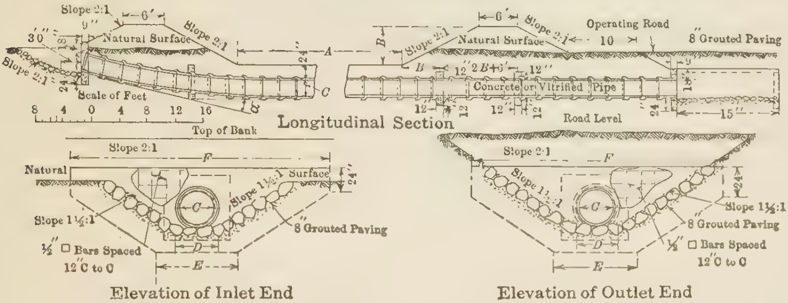

FIG. 87.—Pipe drainage culvert under Dodson North Canal. Milk River Project, Mont.

North Canal of the Milk River project, Montana (Fig. 87). The conduit is 24 or 30-inch concrete or vitrified pipe, and the inlet is of grouted paving formed into a funnel-shaped entrance.

An interesting type of large-size box culvert, combined with a sluiceway or wasteway outlet from the canal, is shown by the structure used at Dry Sheep Creek on the Interstate Canal of the North Platte project, Nebraska (Fig. 88 and Plate X, Figs. A and B). The culvert is designed for a maximum drainage flow of 2,400 second-feet; it is built in four rectangular compartments, and is depressed below the natural ground surface. The inlet has a special conical funnel shape, formed of a ring or weir wall along the edge, with inner warped surfaces. The outlet forms a gradually expanding section lined with concrete 6 inches thick for 80 feet downstream. The wasteway outlet from the canal discharges through an opening in the roof of each compartment and serves the purpose of overflow spillway, wasteway and sluiceway. The canal outlet is divided by piers into four openings, each regulated by flashboards placed in grooves formed in the sides of the piers and by a straight, screw-lift, cast-iron,

ribbed, rectangular gate placed back of the flashboards. With the flashboards the depth of water in the canal may be regulated and the surplus flow passes through the undershot gate opening into the culvert. The flashboards may be entirely removed, and the outlet can then be used for a sluiceway.

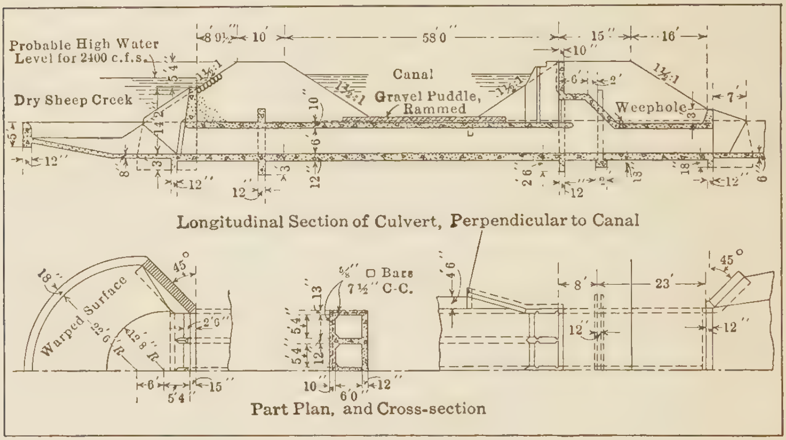

FIG. 88.—Dry Sheep Creek culvert on Interstate Canal. North Platte Project, Neb.

Overchutes or Flumes for Carrying Drainage Water over Irrigation Canals.—This type of structure requires that the bed of the drainage channel or depression be above the full water supply level of the canal. This condition will usually be obtained only for canals constructed on steep sidehills. It is used only occasionally and usually for small volumes of water. On a few projects it has been used to advantage to carry waste water from small laterals over main laterals to supply high land adjacent to the main laterals, which could not be served from the main laterals.

Overchutes are essentially short flumes over the canal, with suitable inlet and outlet structures (Plate X, Fig. C). Where a high velocity at the outlet to the flume is produced by the steep grade or fall at the outlet, the flume may be a chute with the outlet formed as a stilling basin, or may be protected with riprap, paving, or concrete lining. A reinforced concrete overchute to carry Pryor Creek over the main canal of the Huntley project, Montana, is shown in Fig. 89. Pryor Creek is a torrential stream, with a maximum flood flow, since the construction of the flume

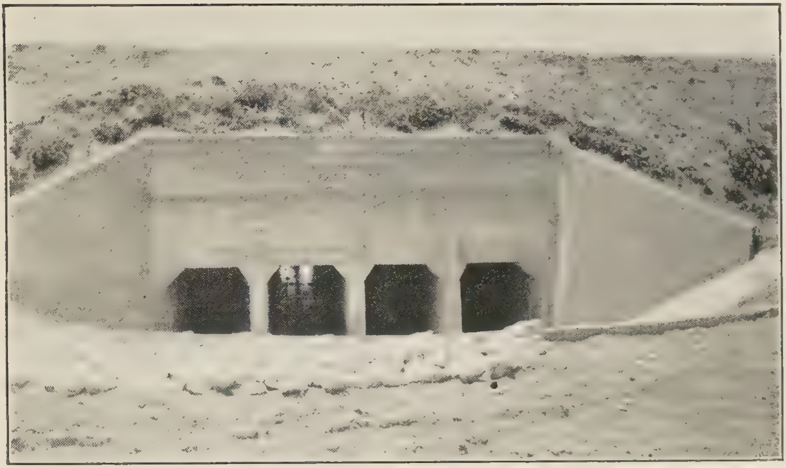

FIG. A.—Inlet of Dry Sheep Creek culvert on Interstate Canal. North Platte Project, Neb.

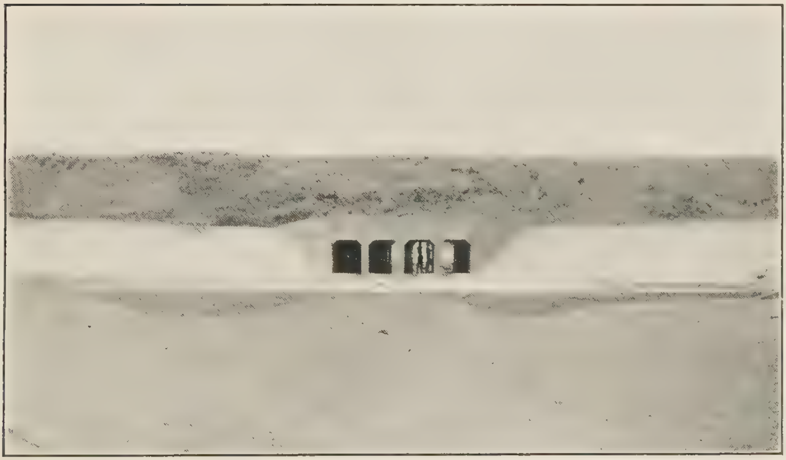

FIG. B.—Outlet of Dry Sheep Creek culvert on Interstate Canal. North Platte Project, Neb.

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PLATE X.

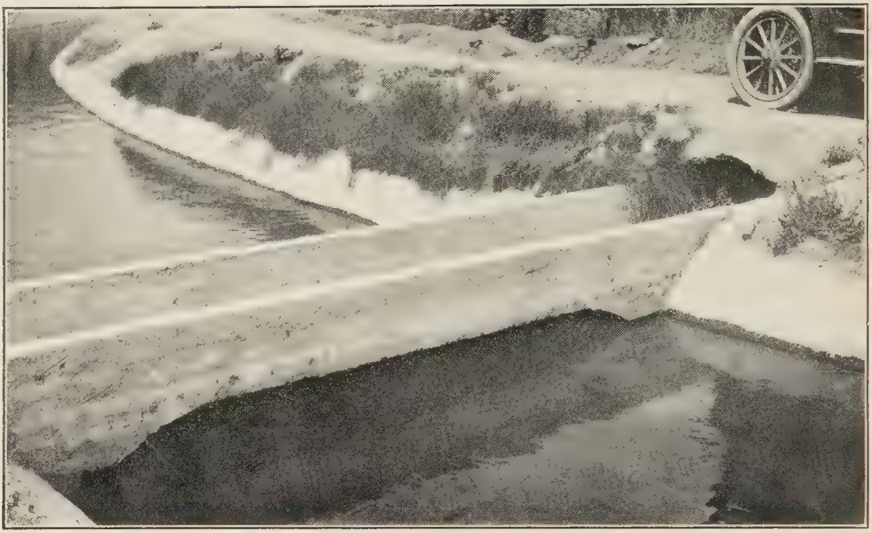

FIG. C.—Storm water concrete flume overchute Gage Canal. Calif.

FIG. D.—Level crossing and wasteway. Umatilla Project, Ore.

in 1906, of 990 cubic feet per second in August, 1909. A design of wooden flume overchute to carry storm water over the East Park Feed Canal of the Orland project is shown in Fig. 90.

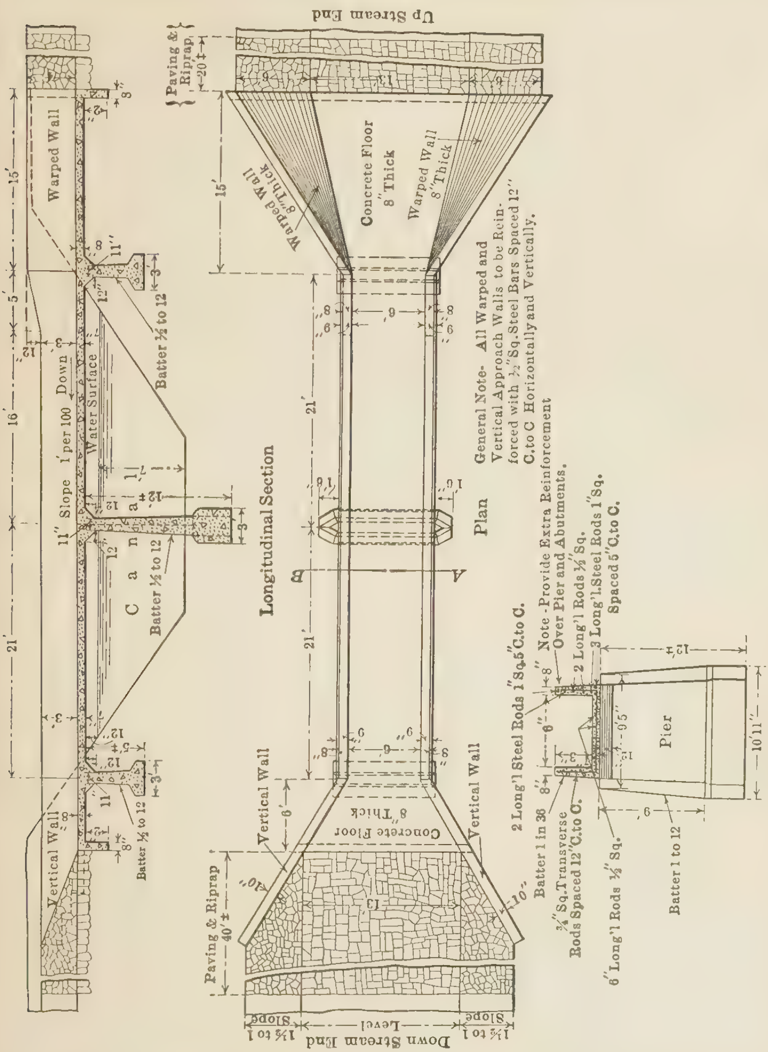

FIG. 89.—Flume for carrying storm water over canal west of Huntley. Huntley Project, Mont.

Level Crossings to Let the Drainage Water in the Canal and in Some Cases through the Canal.—A level crossing is formed by allowing the drainage water into the canal and either carrying it in the canal or passing it through the canal. The following conditions will control the design of the crossing:

1. The drainage water occurs during the irrigation period and at a time when there is a deficiency in the available water supply. In this case the drainage water is valuable in supplementing the deficiency, and at least part of the drainage water can be carried in the canal and the excess wasted through the canal either at a point directly opposite the inlet or at some wasteway below, if the carrying capacity of the canal down to that wasteway is sufficient.

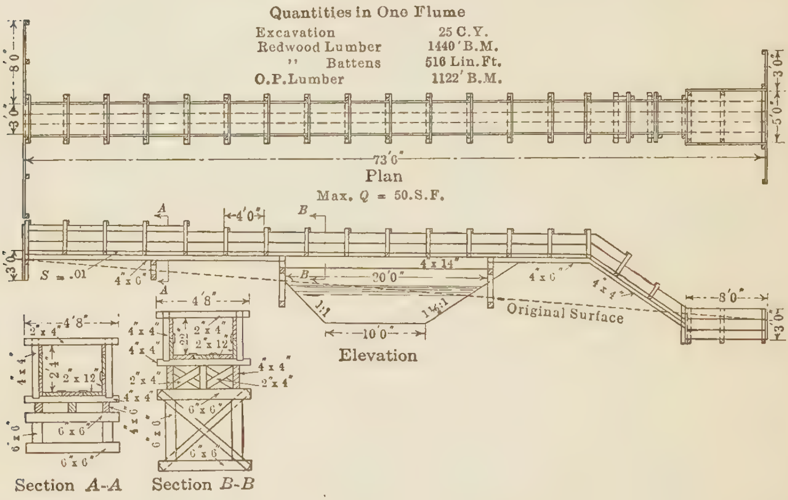

FIG. 90.—Overchute wooden flume on East Park Feed Canal. Orland Project, Calif.

2. The drainage water occurs during the non-irrigating period, and is not in excess of the carrying capacity of the canal. In this case it may be carried in the canal to the nearest wasteway, or may be passed through the canal.

3. The drainage water is far in excess of the carrying capacity of the canal, or occurs in excessive quantities at a time when the canal may be operated at full capacity. In this case the drainage water must be passed through the canal.

A level crossing in its simplest form may be a basin or reservoir, formed by damming the drainage channel or depression, into which the canal water enters and mingles with the drainage water, and from which the combined supply is taken out. The basin may be a small depression, in which the water backs up only a short distance up the channel, such as when formed by using the single downhill bank of the canal for the damming of the channel;

or may be a reservoir of sufficient size to act as a storage and regulating reservoir, or a settling basin into which the sand and silt may be deposited. When used as regulating and storage reservoir, it will usually be desirable to have regulating headgates at the outlet to the reservoir, and in some cases it may be necessary to provide a wasteway. The use of such a reservoir may require the construction of a large dam involving considerable expenditure, which should not be undertaken without a careful consideration of the advantages to be gained and of the suitability of the reservoir site. The experience on a number of projects has been in many respects disappointing, because of the large seepage losses. On at least two reservoir sites in California, formed in the foothill lands, excessive seepage losses were obtained, which materially decreased the value of these reservoirs for storage, the soil in these cases being underlaid in part by sandy strata. On a project in Idaho a foothill reservoir, formed at a great cost, proved to be practically worthless on account of the large seepage loss through the open lava soil and the fissured lava rock with which it was underlaid. Even under favorable conditions, these reservoirs will usually be comparatively shallow and will lose considerably by evaporation. A serious disadvantage of allowing the irrigation water to spread out in a large basin or storage reservoir is that a large part of the flow entering the reservoir will be lost by evaporation and percolation at a time when there may be no inflow from drainage channels to balance it; in some cases it will be feasible to overcome this by using a by-pass canal around the reservoir or by confining the canal between two banks with an inlet from the canal into the storage reservoir or basin and an outlet from the reservoir into the canal.

The usual type of level crossing where the drainage water is to be carried through the canal will consist of the inlet made in the upstream bank of the canal and the outlet made in the downstream bank. The inlet is similar in construction to an overflow spillway. The outlet is either an overflow spillway, an automatic spillway, or a wasteway structure; the considerations determining the selection of the type and the details of design are essentially the same as those given in the discussion of wasteways. The canal bed between the inlet and outlet structures must be protected by paving or concrete lining, and will usually be depressed or formed as part of a channel through the canal, as illustrated by

the level crossing used on the Umatilla project (Fig. 91 and Plate X, Fig. D). In this structure the floor and side slopes are all protected with concrete lining, with cut-off toe walls along the

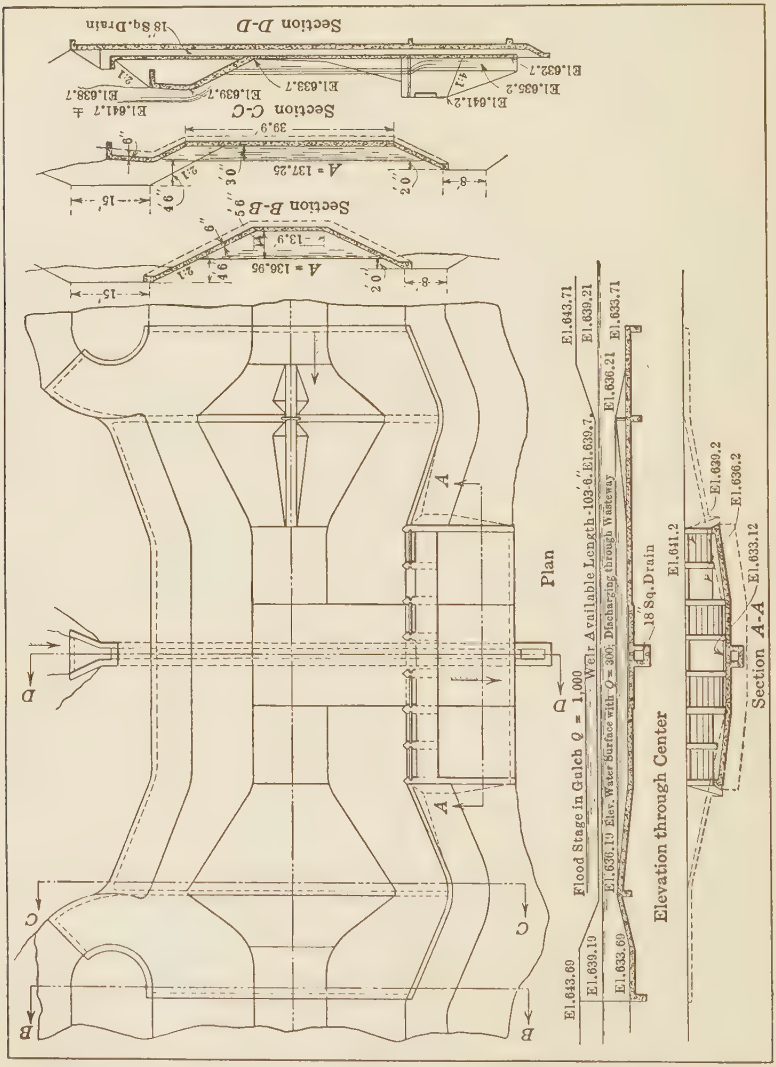

FIG. 91.—Level drainage and wasteway. Umatilla Project, Wash.

edges where necessary to guard against undermining. The depressed basin is formed between two raised sills in the floor of the canal, and of sufficient depth to give a discharge capacity through outlet gates in the bank of the canal of 300 second-feet, when the

water level in the basin is level with the crest of these sills. This permits the discharge of this volume of flood flow through the canal, during the period when the canal carries no irrigation water, without any flood water passing downstream into the canal. The only purpose apparently of the notch in the upstream raised sill is to drain out of the canal the water held back by the sill. If it were necessary to operate the canal with a small flow for domestic water a similar notch would be desirable in the downstream raised sill. The outlet through the canal bank may also be used for wasteway to discharge the entire full supply canal capacity of 300 second-feet, for which purpose the upstream raised sill will then prevent the excessive drawdown of the water level

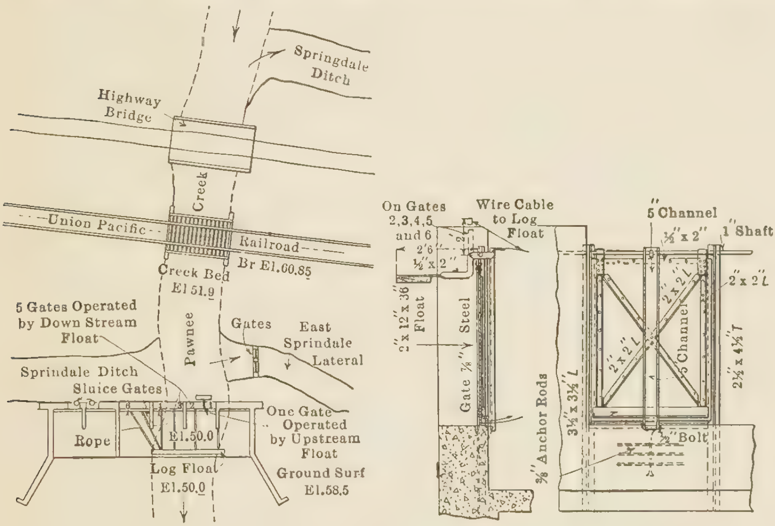

FIG. 92.—Automatic escape for drainage crossing. Springdale Ditch Co., Colo.

in the canal above. The drain under the floor of the canal will discharge smaller flows in the drainage channel.

The same form of structure has been designed for a number of level crossings and wasteways on the Milk River project, Montana, with in some cases automatic balanced gates in the outlet openings through the canal bank, of the type previously described in the discussion of sluiceways and wasteways.

An interesting type of automatic escape for a drainage crossing installed for the Springdale Ditch Co. of Colorado is shown in Fig. 92. The structure is installed at the junction of Spring-

dale ditch, whose capacity is 75 second-feet, with Pawnee Creek. which is ordinarily dry but is subject to sudden periodic floods. As shown in the sketch, the creek channel is used for a short distance to convey the ditch water. The structure has six automatic floodgates and two sluicegates. Each floodgate, made of 3/8-inch steel plate, stiffened with angles placed diagonally and along the edges, is hung at the top to a 1-inch shaft and when released swings out. The gate is held in place by a 5-inch channel, placed on the downstream side, whose lower end is connected with a pivot joint to an anchor below the sill of the gate and whose upper end engages with a catch or tripping device. About 1/3 of the entire water pressure on the gate is transmitted to the shaft on which the gate is hung, and about 2/3 is transmitted to the lower part of the channel very near its pivoting point; because of the large leverage thus obtained, only a comparatively small force is exerted at the tripping connection with the upper end of the channel. The tripping device can therefore be released by a small operating force. For one of the six gates the tripping device is connected to a wooden float on the upstream side, while for the others the tripping devices are all connected to a log float on the downstream side.

Simple forms of inlet for small volumes of drainage waters which may be discharged into the canal and not carried through the canal consist of a shallow cut made in the uphill bank, protected with concrete lining or rock paving extending down the overpour canal slope, over the canal floor and up the opposite canal slope. Fig. 93 shows this form of inlet used on the Lower Yellowstone project, Montana. The inlet may be made with a pipe through the upstream canal bank, but this form is not usually desirable because of the liability of obstruction by débris.

Comparative Merits of the Different Types of Crossings.—A comparison of the different types of drainage crossings to determine the type to be used must depend on the considerations stated above and on a cost comparison.

The fourth, fifth and sixth types which require that the drainage flow be carried either under, over, or through the irrigation canal in channels of a limited size have the disadvantage that except for very minor drainage flows the safety of the structure depends on the volumes of maximum flood flows, the estimate of which is based on a number of uncertainties. The use of level or inlet crossings for the addition of drainage water to the irri-

gation water may in some cases be desirable to supplement the available water supply, but where the drainage water occurs at a time during the irrigation period when it is not desired, the safety of the system is increased, and conditions for operation are more favorable if the drainage water is not allowed to enter the canal.

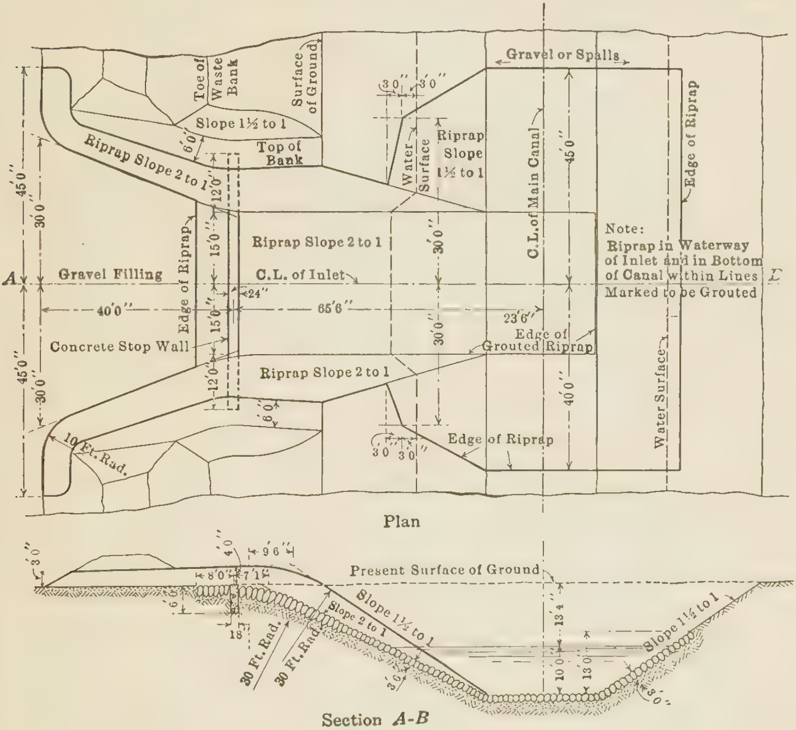

FIG. 93.—Inlet crossing at Coal and Blacktail Coulees. Lower Yellowstone Project, Mont.

The use of inverted siphons to carry the irrigation water across a drainage channel, with the pipe line constructed under the stream bed, leaves the waterway of the drainage channel unobstructed, but there may be certain conditions, previously stated in Vol. II, Chapter X, which may make it preferable to carry the pipe line on a bridge or on piers above the drainage waterway.

CHAPTER VII

DROPS AND CHUTES IN CANALS

A drop is a structure designed to discharge the water in the canal from one level to a lower level by a vertical drop. Usually the drop is formed of: the breast wall across the canal; the inlet wings and floor on the upstream side; the two side walls on the downstream side; the floor and water cushion, at the toe of the breast wall to receive the falling water; the outlet wings and the outlet floor.

A chute is an inclined drop formed by an open canal placed on a steep grade and lined with concrete, wood or sheet steel, so as to resist high velocities, or by a pipe connecting the upper level to the lower level.

These structures are used where it is necessary to adjust a canal to the topography, or where a natural drop occurs in the surface of the ground on the line of the canal, or where an excess in grade must be taken up.

The grade to be given to a canal will depend on the location of the canal, the velocity which the material will stand without erosion, and the form of the cross section. Where a canal is located on a ridge it is desirable if possible to select for the grade of the canal a grade equal to that of the ridge; if that grade is flat the use of a comparatively deep cross section may be necessary to obtain as high velocity as possible; if that grade is steep the use of a shallow and wide cross section may produce a velocity which is not excessive; but the extent to which the velocity can be regulated by changing the form of the cross section is not very great.

The use of a flatter canal grade than the surface grade on the line of the ridge requires that the excess in grade be taken up at certain points on the canal by the insertion of falls or chutes. Directly below a drop the canal will be deep in cut; from this point downstream the depth of cut decreases and the canal bed if continued would have to be carried in embankment; a second drop should be inserted before this point is reached on account of the dangers of breaks where the canal is all in embankment.

FIG. A.—Series of drops on Comanche Canal in hardpan formation.
Arkansas Valley Sugar Beet & Irrigated Lands Co., Colo.

FIG. B.—Series of drops on Comanche Canal looking downstream.
Arkansas Valley Sugar Beet & Irrigated Lands Co., Colo.

(Facing page 222)

FIG. C.—Small wooden drop. Truckee-Carson Project, Nev.

FIG. D.—Reinforced Concrete Drop. Oakley Project, Idaho.

Where the canal is located on a side hill on a line of excessive grade or where it must be located to connect two fixed points whose difference in elevation is greater than can be used for the canal grade, falls are necessary, and when located at favorable points they may be used to advantage in obtaining an economic location. Falls may also be necessary when a small canal is to be enlarged to carry a greater volume of water, when the resulting velocity corresponding to the grade formerly used will be increased to a greater value than the material will stand.

Location of Drops and Economic Height of Drops.—The proper location of drops will depend on the topography. Where the slope of the surface of the ground along the line of the canal is

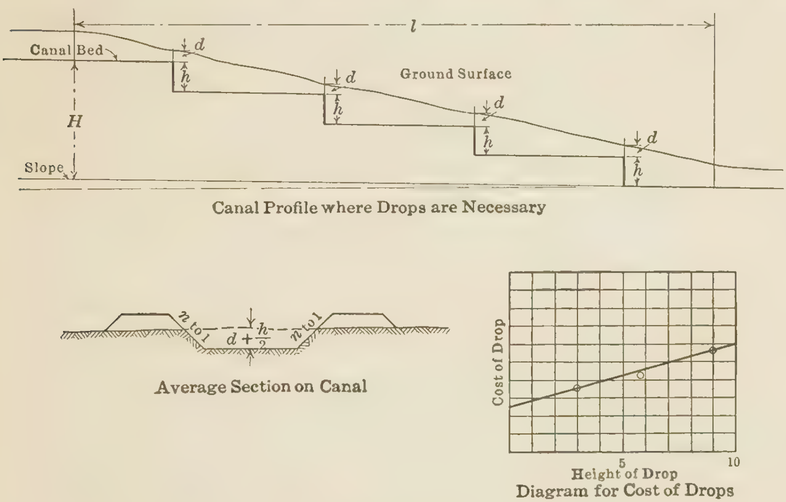

FIG. 94.—Diagrams to illustrate the determination of economic height of drops.

irregular, there will be favorable points such as an abrupt drop, which will indicate the position for a drop. Where the slope is uniform, drops will be spaced at about equal intervals and of about equal heights (Fig. 94). The choice will be between high drops spaced far apart and a greater number of low drops spaced closer together. Low drops spaced close together will give a greater total cost of drops, but a smaller volume of excavation. Unless the excess in grade is very great, low drops will usually be more economical. In any case there is an economic height of drop which will give the minimum total cost. This is determined as follows (Fig. 94):

Let d = minimum depth of cut of canal or cut on upstream side of drop in feet.
b = bottom width of canal in feet.
$n:1$ = side slopes of canal.
A = average area of canal cross section in cut in square feet.
h = height of one drop in feet.
l = total length of canal considered in feet.
H = total excess fall in length l in feet.
V = total volume of excavation in cubic feet.
C_1 = cost of those elements of a drop which are common to drops of any height, such as part of the cost of the wings, side walls, floors, etc.
K = a constant depending on the type of drop.
C_2 = cost of a single drop.
C_3 = total cost of drops in length l.
C_e = cost of excavation per cubic foot.
C_4 = total cost of excavation.
C = total cost of drop and excavation.

The cost of a single drop may be expressed by the equation:

$$C_2 = C_1 + Kh$$

To determine the value of C_1 and K the type of drop must be decided and the cost of drops of various heights estimated. These costs with the corresponding heights are plotted and the points obtained joined approximately by a straight line. The intersection of this line with the cost axis will give C_1 and the slope of the line will give K.

The relations between the different elements are:

$$A = \left(d + \frac{h}{2}\right)\left[\left(b + n\left(d + \frac{h}{2}\right)\right)\right]$$

$$C_4 = lAC_e = lC_e\left(d + \frac{h}{2}\right)\left[\left(b + n\left(d + \frac{h}{2}\right)\right)\right]$$

$$C_3 = \frac{H}{h}(C_1 + Kh)$$

$$C = C_3 + C_4 = \frac{C_1 H}{h} + KH + lC_e\left[\left(bd + \frac{bh}{2} + n\left(d + \frac{h}{2}\right)^2\right)\right]$$

To obtain value of h which will give minimum total cost, take

first derivative of C with respect to h, place the result equal to zero and obtain

$$h^3 + \left(2d + \frac{b}{n}\right)h^2 - \frac{2C_1H}{nlC_e} = 0$$

Principles of Design.—The effects of a drop on the flow of water in the canal and the dynamic forces which must be specially considered in the design are the following:

First.—The effect on the velocity of flow on the upstream side of the drop.

Second.—Effect of force of impact produced by the water fall at the foot of the drop.

Third.—Erosive effect of eddies and irregular currents produced at the outlet to the drop floor or water cushion.

First.—**Effect of Drop on Velocity of Flow on the Upstream Side of the Drop.**—When the water passes over a weir wall or over the raised crest of the breast wall of a drop, beginning at a short distance upstream from the crest of the fall, the water surface begins to drop down, so that the depth of water directly at the crest may have a minimum value of only $\frac{2}{3}$ of the full depth obtained above. This local action increases the velocity, but it extends only a few feet upstream of the breast wall; it cannot be prevented, and if the increase in velocity corresponding to the decrease in depth produces a velocity which is too high, the bed of the canal may have to be protected against erosion by a short floor on the upstream side. Without this floor, a shallow cavity may be washed out on the upstream side of the breast wall. The term depth of water on the crest of a weir or drop, as generally used, refers to the full depth of water measured a short distance upstream from the crest before the point where the water surface begins to drop. A more serious effect occurs when the width or crest length of the drop is so large that the depth of water on the crest required to pass the volume of water in the canal is so small that the drop in the water surface causes a decrease in the water cross-sectional area with an increase in velocity, which may have a marked effect for considerable distance upstream and result in erosion of the bed and banks. This action will not occur if the drop is designed to make the discharge over the drop and the carrying capacity of the canal equal when the level of the surface of the water is the same at the drop as it is in the canal. This may be obtained by three forms of design: (A) By the use of

contracted length of crest (Plate XI, Fig. D). (*B*) By the use of a raised crest (Fig. 95). (*C*) By the use of a notched breast wall (Plate XII, Figs. A and B).

(*a*) **Use of a Contracted Length of Crest of Drop.**—With this form the crest of the fall is level with the floor of the canal; the depth of water on the crest of the breast wall is the same as the normal depth in the canal, and the length is computed for this depth of water and the corresponding carrying capacity of the canal. For this computation the flow is considered as that over a weir, although in this case the weir crest is level with the bed of the canal and gives a weir of zero height. For these conditions of

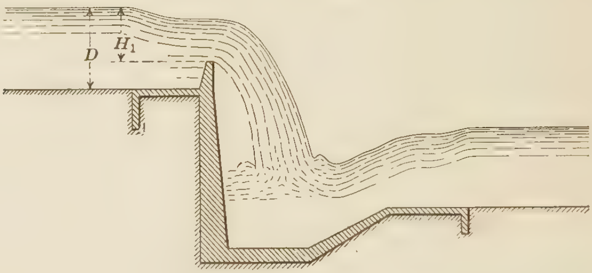

FIG. 95.—Raised crest of breast wall of drop to prevent increase in velocity upstream.

flow there is practically no experimental data to determine the coefficient C in the commonly used weir formula: $Q = ClH^{3/2}$. Bellassis states that for these conditions there is no local surface fall of the water surface such as occurs in ordinary weirs and presents a formula, which reduces to

$$Q = 4.75lH^{3/2}$$

where H is the head on depth of water measured at the edge of the drop.

Bazin's formula for a weir height of zero reduces to

$$Q = 5.03lH^{3/2}$$

These special values of the coefficient apparently permit the use of these formulæ with no correction for the velocity of approach. With low drops the water level on the downstream side may be above the crest of the drop, in which case the conditions are those of a submerged or drowned weir, for which various formulæ may be used. Clemens Herschel's formula has the advantage of simplicity and will give results well within the degree of accuracy

Fig. A.—Reinforced concrete notched drop. Twin Falls Salmon River Land & Water Co., Idaho.

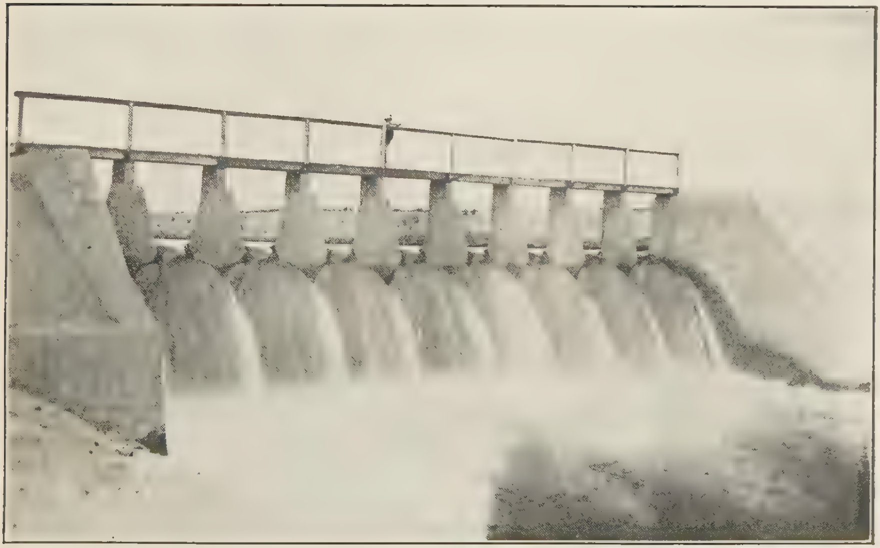

Fig. B.—Same as Plate XII, Fig. A.

(Facing page 226)

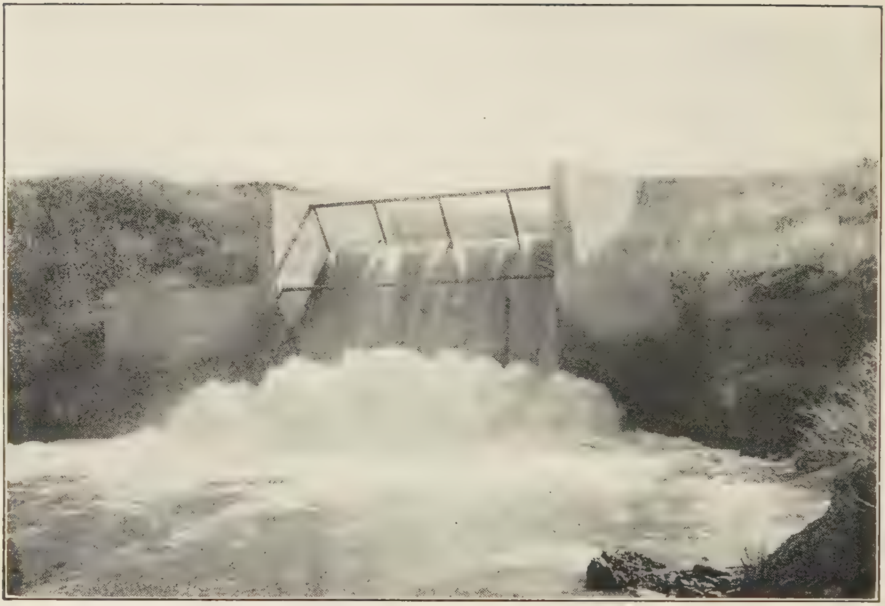

FIG. C.—Fifteen and one-half foot plain concrete drop. Modesto Irrigation District, Calif.

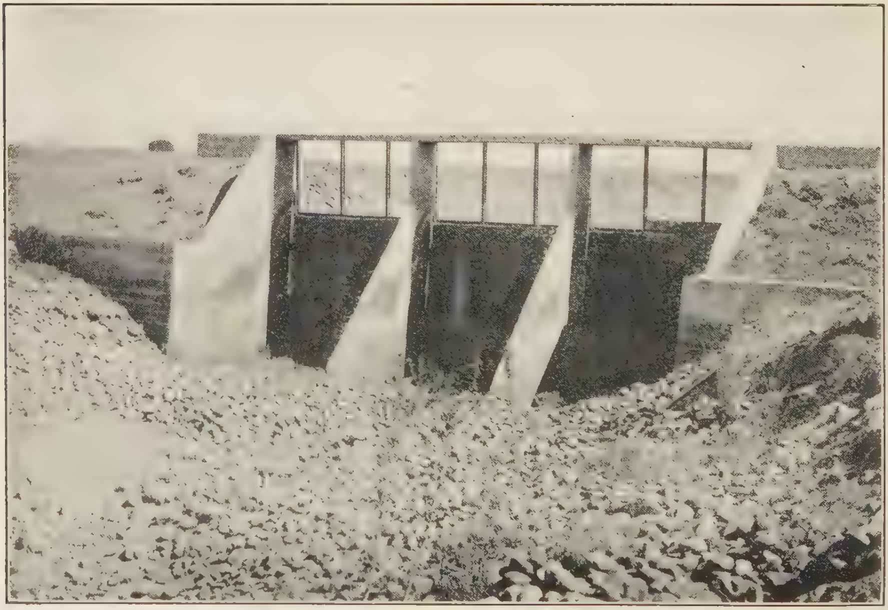

FIG. D.—Reinforced concrete drop. Modesto Irrigation District, Calif.

required. It has the following value for a thin-edge or sharp-edge weir:

$$Q = 3.33l(NH)^{3/2}$$

where H is the head on the crest of the drop and N is a coefficient which depends on the proportional submergence and for which values have been given in the discussion of diversion weirs. The effect of submergence is small unless the depth of submergence is a large proportion of the total depth of water on the crest. A submergence of less than 15 per cent. does not affect the discharge any appreciable extent; submergences of 25 per cent. and 50 per cent. give values of N equal to 0.98 and 0.89, respectively. These results indicate that for a proportional submergence of less than 25 per cent. it would be sufficiently accurate to use the free discharge formula.

To adapt Clemens Herschel's formula to weir crests other than a thin edge weir, the coefficient C used for weirs with free discharge may be substituted for the special value 3.33. The formula may therefore be written:

$$Q = 4.75l(NH)^{3/2}$$

for the submerged weir of zero height.

The disadvantages of the above form of drop are:

First.—The length of crest corresponds to the depth of water in the canal for which it was computed and only works correctly for that depth.

Second.—Length of the crest wall is less than the average width of the canal. This produces a contraction in the cross sectional area of the channel at the inlet to the drop and requires an expansion on the downstream side of the drop which causes eddies, making it more difficult to overcome the resulting erosion at the outlet to the drop and to bring the flow back to its normal condition.

(*b*) **Use of a Raised Crest.**—With this form a length of breast wall greater than the contracted length obtained by the above method is used and the crest of the breast wall is raised to a height which will hold the water level on the crest to the same height as in the canal above. The width of breast wall is usually made equal to the average width of the canal. The height of water H_1 on the crest of the raised sill is then computed for this selected length and for the required carrying capacity (Fig. 95).

For these computations it will be sufficiently accurate to use the simple weir formula, corrected for velocity of approach when necessary. The equations are then:

$H_1 = \left(\frac{Q}{Cl}\right)^{2/3}$ for no end contractions and no velocity of approach.

$H_1 = \left(\frac{Q}{Cl} - h^{3/2}\right)^{2/3} - h$ for no end contraction, with velocity of approach.

h = velocity head $= \frac{v^2}{2g}$, where v = velocity of approach.

If D = depth of water in the canal, then the crest is raised a height above the floor of the canal equal to $H_2 = D - H_1$. The value of the coefficient C may be taken as 3.33 for a thin sharp-edged, raised crest, such as when the raised crest is formed by flashboards placed vertically or for a vertical rectangular wall, 4 to 30 inches thick, with sharp edges when the depth of water on the crest is equal to at least twice the thickness of the wall; rounding the upstream corner on a 4-inch radius, gives a value of about 3.50, and decreasing the depth of water to a value equal to about the thickness of the wall gives values of about 2.90 for sharp corners and 3.10 for rounded, upstream edge. Where the drop is submerged to a depth greater than about 25 per cent., the equations are modified as indicated above.

The objections to this method are:

First.—The height of the raised crest is correct only for the carrying capacity of the canal for which it has been computed; unless it is a movable crest which may be adjusted for any height, such as by the use of flashboards.

Second.—The raised crest, if stationary, holds a body of water in the canal, which cannot be drained and which favors a deposit of silt when the canal is operated at a fraction of its full capacity with a corresponding lower velocity. This can be remedied by making a part or the entire raised crest removable.

Third.—It increases to a small extent the height and length of the breast wall; but the extra cost of the breast wall as compared to that of the contracted length used in the first method may be more than balanced by the shorter length of wing walls required to make connections with the canal.

The advantage of a raised crest drop is that the channel is not contracted, which produces less tendency for erosion by eddies at

the outlet. It is well adapted to locations where the drop is to be used also for a check gate by regulating the inlet with flashboards or gates placed at the crest.

(*c*) **Use of a Notched Breast Wall** (Plate XII, Figs. A and B).—This is obtained by extending the breast wall above the canal bed up to full supply level and dividing it into one or more trapezoidal notches separated by trapezoidal piers. The short base of the trapezoidal notch is level with the floor of the canal and the sides of the notch spread out toward the top. The object of these notches is to regulate the flow over the drop so that for any discharge of the canal, the depth of water on the crest at the notches is maintained equal to the corresponding normal depth in the canal. To obtain equal depths of water at the drop and in the canal for every discharge of the canal would require a notch whose side would not be straight lines but concave curved lines which, however, do not deviate very far from straight lines properly placed. The design of the notched crest wall requires the selection of the total length between side walls, the number of notches and the dimensions of the notch. The selection of the number of notches is not dependent on theoretical considerations, as the entire flow could be regulated with one notch as well as a number of notches; the practical advantage obtained by using a number of notches is the division of the flow into smaller volumes, the discharge of which produces less severe action on the downstream side of the fall. On the other hand, narrow notches may be obstructed by material transported by the water. For the approximate determination of the controlling dimensions, subject to modification based on accurate computations for the notch openings, the following empirical rules have been recommended as representing the practice in India:

1. Make the total length of breast wall between side walls equal to about the bottom width of the canal.

2. The top width of the notches should not exceed the depth of water in the canal and may be from $\frac{3}{4}$ to 1 times the depth.

3. The top length of the crest of the piers separating the openings should not be less than $\frac{1}{2}$ the depth of water.

4. The number of notches obtained from the above conditions is equal to about the bottom width of the canal divided by $1\frac{1}{4}$ to $1\frac{1}{2}$ times the depth of water.

The dimensions of the notch opening which must be obtained are the depth, which is usually equal to the full depth of water in

the canal, the bottom width and the slope of the sides, which are computed from the equation of flow through notches. This equation derived below contains two unknown quantities; the bottom width and the side slope. The solution of the problem is obtained by applying the equation to two special cases, for each one of which the carrying capacity and the corresponding depth of water are known. These special values should be selected to represent the prevalent conditions of flow. The general equation for free discharge through a notch, neglecting the velocity of approach, which has little effect when it does not exceed about 3 feet per second, is derived as follows:

b = bottom width of notch.
$n:1$ = side slopes of notch.
H = full supply depth of water corresponding to full discharge Q.
H_1 and H_2 = depth of water for special cases corresponding to discharges Q_1 and Q_2 and obtained by Kutter Chezy's formula.
C_1 = coefficient of discharge.

The differential discharge for a differential area equal to the width of the notch at a depth y below the top of the notch where H is the depth of the notch is then: $dQ = dyC_1[b + 2n(H - y)]\sqrt{2gy}$ by integration between limits of $y = H$ and $y = 0$.

$$Q = \frac{2}{3}\sqrt{2g}C_1(bH^{3/2} + \frac{4}{5}nH^{5/2}) = 5.35C_1(bH^{3/2} + \frac{4}{5}nH^{5/2})$$

When the velocity of approach is not neglected, the formula takes the more complicated form:

$$Q = 5.35\mathrm{C}_1\left\{b\left[(H + h)^{3/2} - h^{3/2}\right] - 2n\left[-Hh^{3/2} - \frac{2}{5}\left\{(H + h)^{5/2} - h^{5/2}\right\}\right]\right\}$$

When the notch is submerged the formula for flow may be derived by considering the notch in two parts. The discharge of the part which is submerged is equal to that of an orifice whose area is equal to the submerged area of the notch, and the head is taken as equal to the difference in elevation of the water levels. The discharge of the part which is above submergence or above the downstream water level is equal to that of a free fall for a notch whose dimensions are those above the plane of submergence.

Neglecting the velocity of approach, the entire discharge for the notch is obtained by a formula which reduces to:

$$Q = 5.35C_1\sqrt{H - H_s}\left\{b\left(H + \frac{H_s}{2}\right) + 2n\left[\frac{3H_s^2}{4} + H_s(H - H_s) + 0.4(H - H_s)^2\right]\right\}$$

Where H_s is the depth of submergence above the base of the notch or the height of submergence, and $H - H_s$ is the difference in water level.

The coefficient of discharge C_1 for notch drops on the Sirhind Canals in India was found by Mr. Burton to range from 0.662 to 0.676 where the corrections for velocity of approach were made. Mr. A.G. Reid in Punjab Irrigation Branch Paper No. 2, on notched falls states, that if the velocity of approach be neglected the following values should be used: $C_1 = 0.70$ for notches on distributary canals $= 0.78$ for notches on main canals. With the lower value neglecting the velocity of approach and for free fall the equation becomes:

$$Q = 3.75\left(bH^{3/2} + \frac{4}{5}nH^{5/2}\right)$$

Applying this equation to two special cases, the results are:

$$A = \frac{Q_1}{3.75} = bH_1^{3/2} + \frac{4}{5}nH_1^{5/2}$$

$$B = \frac{Q_2}{3.75} = bH_2^{3/2} + \frac{4}{5}nH_2^{5/2}$$

Solving for b by equalizing the values for n, the result is:

$$b = \frac{BH_1^{5/2} - AH_2^{5/2}}{H_2^{3/2}H_1^{5/2} - H_2^{5/2}H_1^{3/2}}$$

Substituting the value of b in either of the above equations will give the value of n.

It will usually be sufficiently accurate to neglect the velocity of approach. For more accurate results the dimensions may be obtained in the same manner by using the formula in which the velocity of approach is included.

The dimensions of the notch are obtained by the computations based on two depths of water with their corresponding discharges. When the notch is submerged for both depths of water, the formula for submerged flow must be used for the two depths selected.

When the notch is submerged for the greater depth selected and not submerged for the smaller depth, the solution requires the use of the formula for submerged flow for the greater depth and that for free flow for the smaller depth.

In all cases the best depths to be selected are those which will give a trapezoidal notch conforming as nearly as possible to the notch with curved sides theoretically correct for all depths. The depth of water will vary from the larger depth corresponding to the full supply capacity of the canal to the smaller depth corresponding to the minimum working capacity of the canal. The form of notch which will best fit the variable flow between the above limits is obtained when the dimensions are computed for depth H_1 and H_2, representing respectively the height of water level below the full supply level by ¼ the fluctuation in water level, and the height of water level above the low working level by ¼ the fluctuation.

If H = full supply depth of water in canal
H_0 = low working depth of water in canal
then $H_1 = H - \frac{1}{4}(H - H_0)$
$H_2 = H_0 + \frac{1}{4}(H - H_0)$.

Where there are no diversions from the canal above the drop, the notches will regulate the water level so as to prevent any appreciable drop down in the surface of the water, but where there are intermittent diversions through lateral gates above the drop the regulation is not so close. The diversions will cause fluctuations in the flow of the canal, which, when the notch dimensions are determined for a full depth of water corresponding to the maximum flow, with the takeout gates shut, will cause a drop in the water level at the points of diversions, resulting in an increased velocity upstream, with the possibility of erosion and making it more difficult to deliver full supplies through the gates. To prevent this, the notch could be designed for a maximum flow equal to the full supply capacity at the drop when the gates are opened, but this may not be desirable, as it may result in the backing up of the water to a dangerous height in the canal above the drop, when the canal carries the full capacity with all lateral gates shut. The better practice is to design the notches for the full maximum capacity of the canal and to provide for regulation of the water level by raising the base or sill of the notch with stop planks or flashboards.

The above conditions will frequently occur, because a drop is usually located at a point on the canal line where the water cross section on the upstream side is mostly entirely in embankment and this is favorable for the location of lateral takeouts near the upstream side of the drop.

The low working depth of water in the canal is best obtained from a knowledge of the smallest capacity it is practicable to operate the canal; empirical values of not more than $\frac{1}{2}$ nor less than $\frac{1}{3}$ the full supply depth have been recommended by A. G. Reid in his paper on Notched Falls.

To complete the notch a semicircular lip, flush with the base of the notch projects horizontally on the outlet side of the opening.

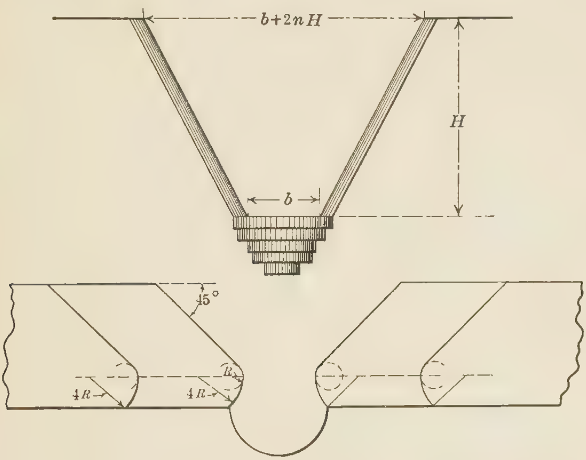

Fig. 96.—Details of typical notch for drop. India.

ing. The object of this lip is to spread the water in a fan-like shape to decrease the force of impact and erosive power of the falling water. The disadvantages of a notch drop are the greater difficulty and cost of construction and that when the conditions of flow are such as to require the regulation of the water level by the insertion of horizontal flashboards at the base of the notch this interferes with the correct action of the notch opening and may destroy the effect of the lip. These conditions frequently exist on laterals where the drop is to be used also as a check gate; as is often the case when the flow of water is variable and it is necessary to maintain the level at a height sufficient to deliver the required volume through lateral headgates on the upstream

side of the drop. Where such conditions exist, the regulation may be obtained with a drop having a breast wall equal in length to about the average width of the canal, with a removable or adjustable raised crest; usually formed by dividing the length of the breast wall by piers or frames into sections or panels, forming openings of such sizes which may be regulated by horizontal flashboards or gates.

The details of a standard form of notch opening for drops on large distributaries in India, according to Lieutenant Colonel J. Clibborn, are shown in the accompanying drawing (Fig. 96). The vertical plane of the profile of the notch is from 6 inches for small to 18 inches for large drops from the downstream face of the breast wall; this represents about 1/3 or a little less than the thickness of the piers or projections of the breast wall which separate the openings. The edges of the piers are curved with arcs of circles on the downstream and upstream side; the center of these circles are all in the vertical plane of the profile of the notch.

Second.—**Effect of Force of Impact Produced by the Water-fall at the Foot of the Drop.**—This effect is less severe with notched drops than with the other forms of drops because of the fan-like spread given to the water by the lip placed at the base of the notch. With a raised crest drop the height of fall is increased a small amount, but the smaller depth of water on the crest and the longer crest length produce less impact force and less tendency for erosion by eddies than with a contracted crest. Three methods have been used to resist the force of falling water: (*a*) The use of a strong floor without a water cushion. (*b*) The use of a floor protected with a water cushion. (*c*) The use of a baffle wall or gratings placed above the floor across the path of the falling water to break the fall of the water. The use of a floor protected by a water cushion is the usual form of construction.

(*a*) **Floor and Water Cushion.**—The length and thickness of the floor and depth of the water cushion will depend on the character of the foundation and the material used for construction and on the force of impact of the falling water, which is proportional to the height of fall and the discharge. Wood will resist impact and erosion better than concrete, especially when the water carries sand or gravel, and a wooden floor bolted to a concrete floor may be used to advantage to protect the concrete, although a water cushion will usually be more desirable. The water cushion is usually a basin depressed below the downstream

bed of the canal, but may be an elevated basin above the canal bed, formed by a cross wall at the downstream end of the basin; while it decreases the required height of side walls and breast wall, it creates a secondary smaller fall, which is not desirable. The length of the water cushion should be at least sufficient to receive the sheet of falling water at about its center; this may be computed by a consideration of the equation of the path of falling water in the same manner as for diversion weirs (page 36) described. The equation reduces to the following form:

$$x = 0.385C\sqrt{Hy}$$

where H is the depth of water on the crest of the drop measured a few feet upstream, which may be corrected for velocity of approach.

C is a coefficient used in the formula for discharge over weirs.

x = horizontal distance at which water strikes when falling through height y.

The coefficient C will be about 3.33 for a sharp-edged raised crest, and about 4.75 for a drop with crest flush with bottom of canal. Using a value of 4.00, the length of the water cushion L will then be

$$L = 2x = 3\sqrt{HF}$$

where F is the fall measured from the upstream water level to the downstream water level.

Where the area of waterway at the crest is divided into openings or panels separated by buttresses or frames for the insertion of flashboards to use the drop as a check gate or to regulate the height of the raised crest, the above formula also applies. When the panels are regulated by gates which discharge the water through the opening under the lower edge of the gate, the velocity will be that through an orifice and the distance out that the sheet of falling water will strike the downstream water level will be maximum when the gate is nearly closed. The head on the center of the opening may be taken as equal to the full depth of water in the canal above, and the height of fall approaches the difference in elevation between the center of the opening and the water level in the water cushion. The maximum velocity will then be approximately $v = \sqrt{2gH}$; the corresponding equation of falling water curve is $x = 2\sqrt{Hy}$, which, for the maximum height of fall, gives a horizontal distance from the crest of the breast wall

to the point where it strikes the water cushion equal to $2\sqrt{HF}$. If the length of the water cushion was obtained for the full flow of the canal passing over the crest of the drop, not obstructed by the gates, it would receive the falling sheet of water of the smaller flow delivered under the gates at a distance above the downstream end of the cushion equal to $\sqrt{HF}$ or about $\frac{2}{3}$ of its length. This margin is considered sufficient, as the volume of water thus discharged is comparatively small and will produce a much smaller impact than the full flow for which the water cushion is designed. A number of empirical rules have been suggested, but it is believed by the writer that the above method of deduction is more logical.

The depth of the water cushion varies widely in practice. It will usually average about $\frac{1}{3}$ of the height of fall measured from the upstream to the downstream water level. A more logical rule which would take into consideration not only the height of fall but also the discharge is obtained by making the depth of the water cushion equal to $\frac{1}{6}$ the length given by the above formula, or about $\frac{1}{2}\sqrt{HF}$. Where the water cushion terminates at the downstream end with a vertical wall, at right angles to the floor, this obstruction produces eddies which makes it more difficult to obtain a regular flow at the outlet; this action can be decreased by sloping up the downstream $\frac{1}{2}$ or $\frac{1}{3}$ of the floor of the water cushion to bring it level with the bed of the canal.

The floor of the water cushion of a concrete drop is usually made 12 inches thick, reinforced both ways with $\frac{3}{8}$ to $\frac{5}{8}$-inch bars, spaced 12 inches apart. There is no uniform practice regarding the placing of this reinforcement; it is placed near the upper face or lower face of the floor or halfway between. Where the floor is on rock a thickness of 6 inches without reinforcement may be sufficient. The floor of drops of small height and for small volumes of water, such as used on small laterals, may be made 9 inches thick, but seldom less than this, and must be reinforced with some form of wire mesh. For very large drops and where the foundation is not firm soil, it may be desirable to increase the thickness to 18 or 24 inches. Where a portion of the floor forms part of the footing of buttress walls, breast wall or side walls, the design of these walls and of their footings may determine the thickness and reinforcement of the floor.

Floors without water cushion are seldom used in the United

States, except for comparatively small drops or where the bed of the canal on the downstream side is on firm rock. Mr. W. G. Bligh, retired executive engineer of the Indian Public Works Department, states that the use of water cushions is not necessary with a notch drop and that it adds to the expense, as it increases the height of the breast wall and side walls. He recommends that the thickness of masonry floors, formed of durable stone set in mortar and grouted with cement, be made equal to $\sqrt{H + F}$ and the length equal to $2(H + F)$ where H is the head of water or depth of water on the crest of the drop and F is the fall measured from the crest of the drop or base of notch to the floor. The thickness obtained by this equation will give results several times greater than that used for reinforced concrete drops, constructed without water cushions in the United States. For low, small drops this thickness is made about 12 inches and for the larger drops seldom exceeds 18 inches.

(*b*) **Baffle Walls and Gratings.**—The use of baffle walls or gratings placed above the water cushion across the path of the falling water for the purpose of breaking the fall of the water has been used in very few cases and is not usually desirable. It is illustrated by the drop with baffle wall on the Comanche Canal in Colorado described farther. Another example is a drop built by the Reclamation Service on the Uncompaghre Canal in Colorado, where the grating was formed of 40-pound rails, spaced 8 inches apart, placed above the water cushion. It was found that this grating produced vibrations which made it necessary to remove them. Another form of grating which was used for canal drops in Upper India, but has now been abandoned, consisted of tapered, narrow, wooden beams, with the broad ends placed on the crest of the drop and sloping upward on the downstream side of the crest, with the narrow ends extending up to full supply water level. These beams form an arrangement similar to the teeth of a comb and divide the flow into a large number of smaller streams, thus decreasing the force of impact of the falling water and holding up the water level on the upstream side. The action is similar to that of a large number of narrow notch openings and is different from that of a baffle wall or gratings placed across the path of falling water. The design and spacing of the openings formed by the tapered beams is fully discussed in P. J. Flynn's book on Irrigation Canals and Other Irrigation Works. The main objections to this type of structure

are the difficulty of construction and the narrow openings which would probably be easily obstructed by the material carried by the water.

Third.—**Erosive Effect of Eddies and Irregular Currents Produced at the Outlet to the Drop Floor or Water Cushion.**—The water cushion and floor resist the impact of the falling water and breaks up its force, but does not destroy irregular currents and eddies whose erosive action is active for considerable distance below the downstream edge of the water cushion. The extent of this erosive action will increase with the height of fall and the degree to which the water area is contracted by the drop. The necessity of protection will depend on the character of the material in which the canal is formed. In some cases where the material is rock or firm soil, no protection is made, but ordinarily some protection is needed. This protection consists of a revetment or lining of the canal, bed and sides, with brush, riprap, or concrete. For average soil conditions this protection should extend below the downstream end of the water cushion for a distance equal to at least the length of the water cushion. When made of concrete, this lining is usually 3 to 6 inches thick and preferably reinforced with a wire mesh or other light reinforcement. To avoid cracking due to settlement, it is necessary that the concrete lining be placed on the undisturbed earth surface or on thoroughly compacted soil. The end of the lining terminates in a cut-off wall which extends into the earth a depth equal to about ½ the depth of water for an average clay loam and to the full depth of water for a loose sandy soil.

DETAILS OF DROPS AND PRINCIPLES OF STRUCTURAL DESIGN

The parts of a drop are:

(*a*) The breast wall.

(*b*) The upstream floor placed at the level of the canal bed.

(*c*) The water cushion and downstream floor.

(*d*) The side walls and wing walls.

(*e*) The upstream and downstream cut-off walls.

(*f*) The protection of canal beds and sides at the outlet to the water cushion.

A drop may include all of these parts or may be formed with some of the parts omitted. The simplest form of drop consists of: the breast wall, extending well on each side into the canal

banks to act also as cut of walls and a short section of lined canal on the downstream side of the drop, formed by a floor and sloping side walls. This type of drop without a water cushion is illustrated by the notch drop of the U. S. Indian Reclamation Service (Fig. 103). To form a water cushion the upstream end of the floor next to the breast wall should be placed below the grade of the canal bed and may slope up to be flush with the canal bed at the downstream end. To prevent underwashing at the downstream end of the floor and side walls, a cut-off wall should extend into the canal bed and banks of the canal. The sloping side walls take the place of the usual side walls and wing walls, giving a more economical drop. In order to prevent cracking due to settlement, the concrete side walls should be built on thoroughly settled slopes; the transversal reinforcement should be continuous with that in the floor and the longitudinal reinforcement in floor and side walls should extend into the breast wall to give the required connection.

The structural design of the parts of a drop involve a consideration of the pressures on the several walls and their design according to the resulting stresses.

The breast wall is designed as a retaining wall. When the upper edge of the wall terminates at the canal bed it is designed to resist the earth pressure from the crest down to the floor of the water cushion. When the breast wall extends above the canal bed, as in raised crest drops, notched drops, or by piers extending up to the upstream water level, the wall must be designed for the hydrostatic pressure as well as for the earth pressure. When the piers or frames extending up to the water level form the support for gates or flashboards, the hydrostatic pressure on the gates or flashboards must be added to the pressure on the piers themselves. The most economical design for low breast walls is usually that of a cantilever wall fixed at the bottom. High breast walls may be more economically designed as a vertical slab supported at the top to a beam formed in the crest of the wall and fixed at the bottom to the floor, or may be designed as a buttressed retaining wall. The most economic design for comparatively high narrow breast walls may be that of a beam slab supported at the two ends to the side walls, resisting a pressure varying in intensity from a maximum at the bottom to a minimum at the crest. The piers of a notched drop are usually designed as cantilevers. When the last three forms of design are used, it is important

that the tendency of overturning of the entire structure be investigated and that the breast wall with the wings and side walls be connected to act together and give a total weight which, with the earth pressure, will make the resultant fall within the middle third of the floor.

The design of the walls may be controlled not by the ordinary earth pressure but by the semi-liquid mud pressure obtained when the structure is backfilled by puddling. The final earth pressure when the backfill is consolidated may be taken as a fluid pressure of 40 pounds per cubic foot, while for the puddle it should be taken as a fluid pressure of 65 pounds per cubic foot. The maximum loading for the breast wall, when the drop is used as a checkgate, may be obtained with a combination of the water pressure on that part of the breast wall extending above the canal bed and the consolidated earth pressure below.

The upstream floor prevents the erosion or washing out of a pocket on the upstream side of the crest of the breast wall, which may result from the increased velocity due to the local drawn-down curve or drop in the surface of the water as it passes over the crest of the drop. This action only extends a short distance. A length of floor equal to the depth of water in the canal will usually be sufficient; it is frequently omitted without harmful results. When made of reinforced concrete, it will usually be 6 to 9 inches thick.

The downstream floor, water cushion and the outlet protection have been previously discussed. The side walls are usually parallel with the direction of flow; they are connected at the bottom to the floor of the drop, are braced across by the breast wall and joined at the ends to the inlet and outlet wings. They may be designed either as cantilever walls fixed at the bottom, or as beam slabs supported at one end against the breast wall and fixed at the other end to the wing wall, provided the wing wall has sufficient anchorage, or as buttressed walls with the buttresses usually on the earth side, or for narrow drops may be braced across by tie posts or braces extending from one side wall to the other across the drop; the economic design will depend on the height and length of the side walls.

The inlet wings with the side walls and outlet wings enclose an earth wedge which prevents the water washing around the structure; it is therefore necessary that the inlet wings extend well into the banks. For small drops the outlet wings are sometimes omitted. The inlet wings are placed either at right angles

or on an angle of 30° to 45° to the axis of the canal or may be formed as warped surfaces. For an equal quantity of material a right angle wing will extend farther in the canal bank and will be subject to a smaller earth pressure. Warped wings or wings placed on an angle guide the flow into the inlet, without checking it to the same extent as right angle wings; this action of right angle wings is, however, not objectionable in a drop. Wings at right angles have a greater tendency to produce eddies at the outlet and for this reason are less desirable. The wing walls should extend below the canal bed down to about the same depth as the cut-off apron at the end of the floors; it is poor economy to step up the lower edge of the wall toward the outer end in the bank.

The cut-off apron at the upper edge of the upstream floor and at the end of the downstream floor should extend below the bed of the canal to a depth of at least ½ the depth of water in the canal for ordinary soil and deeper for loose open soil.

The above principles of design are illustrated by the following examples:

Contracted Crest Plain Concrete Drop on Main Canal of Modesto Irrigation District, California (Fig. 97 and Plate XII, Fig. C).—This structure, built in 1902, was probably one of the earliest uses of concrete for irrigation structures in the United States. It is built for a maximum canal carrying capacity of 630 cubic feet per second and for a height of fall of 15½ feet. The breast wall has a length between side walls of 24 feet, which is less than ½ the average width of the canal. The water area at the crest of the breast wall is divided into four openings by sloping T-beams, imbedded at their lower end in the crest of the breast wall and supported at their upper end on an 8-inch I-beam, extending across the drop and acting as a brace for the side walls. The crest of the breast wall may be raised by flashboards placed with their ends against the T-beams. To carry the maximum discharge, the depth of water on the crest is about 5 feet. The water cushion is formed above the canal bed by a cross wall which produces a secondary fall. The fall to the water surface of the water cushion is 11 feet 6 inches, and the length of the water cushion is only 12 feet which is short for this height of fall and depth of water on the crest. The depth of water cushion of 4 feet is about correct for that height

of drop. The shock of the secondary fall is resisted by the outlet floor.

The structure is entirely non-reinforced; the walls, especially the side walls, are thinner than the usual gravity section required

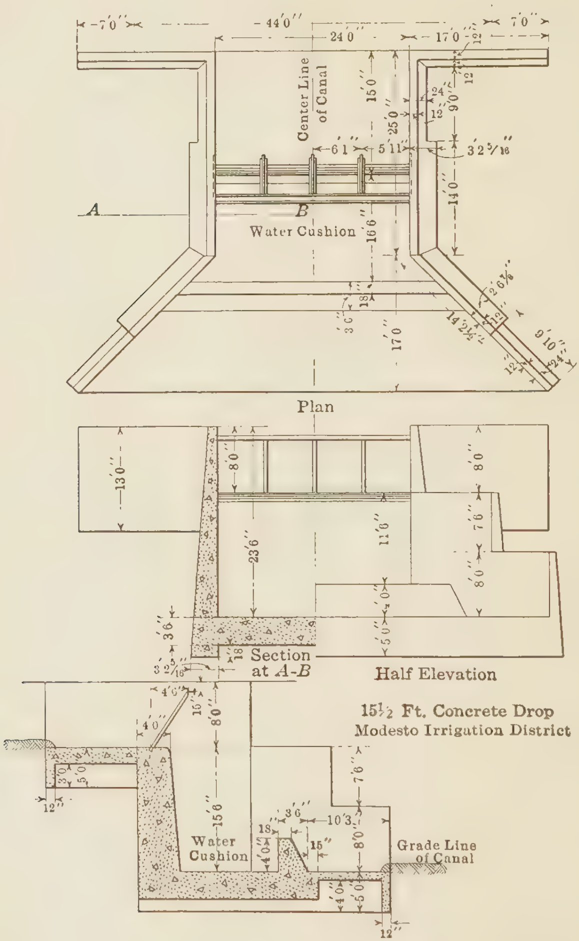

FIG. 97.—Concrete drop. Modesto Irrigation District, Calif.

to resist earth pressure, but the stability of the separate walls is increased by the buttressing effect of the junction of walls and by the connections with the floor. A small tensile stress in the walls is obtained, which is permissible. A comparison

of this drop with reinforced concrete drops indicate that it cost about 40 per cent. more than a reinforced concrete drop.

The following defects in design and methods of improvements are apparent:

The drop has a contracted crest which is not considered desirable, especially as the crest is also arranged for regulation with flashboards. The contraction has a tendency to produce eddies or cross currents around the end of the outlet wings, which has required protection with stone riprap. The raised water cushion with a secondary drop is not desirable and the length of the water cushion is too short. A water cushion depressed below the canal bed, about 5 feet deep and 25 feet long, would give a better structure. The floor of the water cushion need not be thicker than 18 inches and the downstream ½ of it should slope up to bring the downstream edge level with the canal bed. The outlet should be protected with a concrete lining extending beyond the end of the water cushion for a distance of about 20 feet. The upstream floor need not be thicker than 9 inches.

Adjustable Raised Crest Drop on Main Canal of Modesto Irrigation District (Fig. 98 and Plate XII, Fig. D).—The breast wall of this structure has a total crest length of 42 feet, nearly equal to the bottom width of the canal; it is a buttress wall formed by the two side walls and two intermediate buttresses which divide it into three panels each 14 feet long. The buttresses extend above the crest wall and support at the top, above full supply water level, a foot walk. The openings above the crest between the buttress walls are further divided by I-beam brackets into smaller openings, 4 feet 8 inches wide, regulated by means of flashboards.

The height of fall from the crest of the concrete breast wall, level with the upstream elevation of the canal bed, to the downstream elevation of the canal bed is 9 feet; this height may be increased by raising the crest by the insertion of flashboards. The water cushion, about 15 feet long, is formed on a concrete floor 12 inches thick, protected by a wooden floor bolted to the concrete. The floor terminates with a cross wall with its crest 18 inches above the concrete floor. From the crest of this wall there is a raise to the canal bed of 2 feet, giving a total depth of water cushion of about 3½ feet. The upstream and downstream wing walls are at right angles to the structure and extend well into the banks to act as cut-off walls.

The entire structure except the floor is reinforced. The footings and floor were first constructed. To make the joint

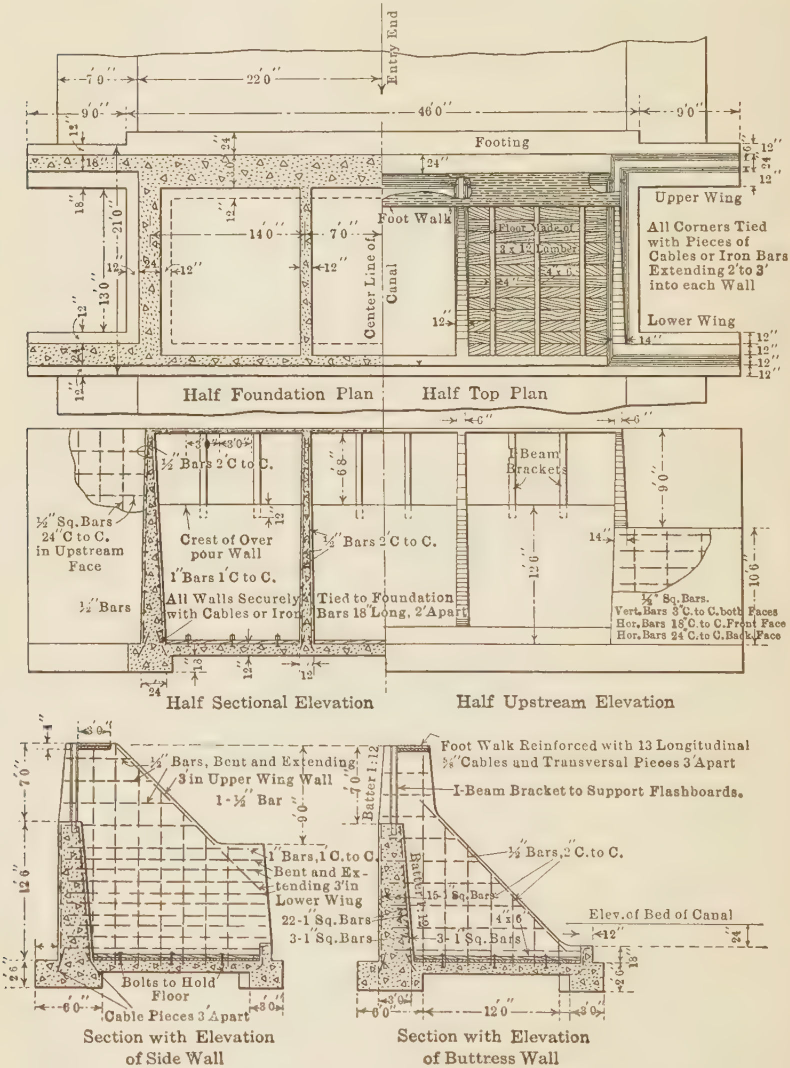

FIG. 98.—Reinforced concrete drop. Modesto Irrigation District, Calif.

with the foot of each wall, pieces of old cable and steel from 18 inches to 3 feet long, spaced about 18 inches apart, in double rows, one for each side of wall, were imbedded at least 9

inches in the floor or footing. All walls are reinforced with square steel bars and the corners securely tied. After the concrete was put in place the structure was carefully backfilled by holding water above the lower cut-off wall, and the excavated material was thrown in and well puddled. The placing and the design of the reinforcement could be improved. The main reinforcement in the breast wall is the horizontal reinforcement next to the downstream face to give the reinforced beam action between the buttresses as points of support. To take care of the negative bending moment, this reinforcement should be brought to the upstream face at the buttresses by bending at least part of the rods. The vertical bars next to the upstream face can only serve to take up temperature stresses.

The dimensions and proportions of the drop are in general correct for this type of drop. There is no upstream floor which is not a serious objection; the design would, however, be improved by lining the canal upstream from the crest of the drop for a distance of 6 to 8 feet with concrete, 3 to 4 inches thick, with a light reinforcement. The lower half of the water cushion floor should be sloped up to be level with the canal bed and the outlet protected for a distance of 15 to 20 feet with a concrete lining on the floor and sides, terminating with a cut-off apron or wall. The cost of the structure is given below:

Excavation	$151.50
Gravel (including hauling)	280.00
Cobbles (including hauling)	170.65
Cement, 205 barrels, @ $2.75	563.75
Hauling cement	62.50
Hauling lumber, etc.	150.00
Miscellaneous team work	38.65
Labor—concreting, riprapping	544.45
Labor—building form, placing steel	150.00
Labor, removing forms, plastering	40.00
Puddling	134.10
Storing lumber and outfit	36.00
All steel (excluding cable), 9,182 pounds of corrugated bars, 830 pounds of steel shapes	329.65
Cook-house supplies	280.15
Blacksmithing, rods, bolts, shovels	29.25
	$2,960.65

Volume of concrete—floor and foundation	86.5 cubic yards
Superstructure	108.5 cubic yards
	195.0

Total cost of drop in terms of concrete used = $15.20

Drop with Baffle Wall, Comanche Canal, Arkansas Valley Sugar Beet and Irrigated Land Co., Colorado (Fig. 99 and Plate XXIII, Figs. A and B).—The structure was built in 1907 on a canal which has a carrying capacity of 450 cubic feet per second. The drop has a width between side walls of 18 feet, and the height of fall is 9 feet. The upstream wing walls run into the banks of the canal on an angle of 30° to the axis of the canal and extend 6 feet 6 inches below the grade of the canal. The downstream wing walls make an angle of 45° with this axis, and extend below the grade of the canal to a depth of 6 feet. To break the force of the falling water before it strikes the water cushion, a baffle wall is placed at an angle across the path of the falling water and acts also as a strut or

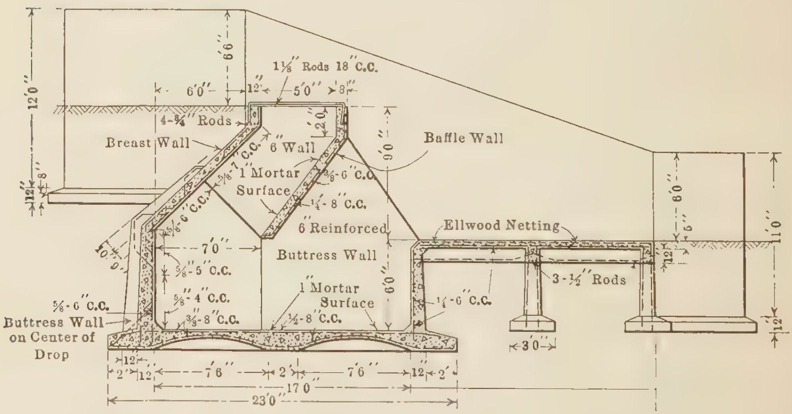

FIG. 99.—Drop; 18 ft. wide. Comanche Canal, Colo.

brace to the side walls. The baffle is supported on the center line of the drop by a buttress wall which extends above the baffle wall to support also the upper sloping part of the breast wall. The top of the baffle wall is tied to the crest of the breast wall with 1⅛-inch rods spaced 18 inches apart. After striking the baffle wall, the water falls in the water cushion, which is 6 feet deep and 17 feet long. The side walls and floor extend for 16 feet beyond the downstream end of the water cushion and join to the outlet wings and cut-off apron, forming a suitable outlet.

The walls are comparatively thin and well reinforced. The

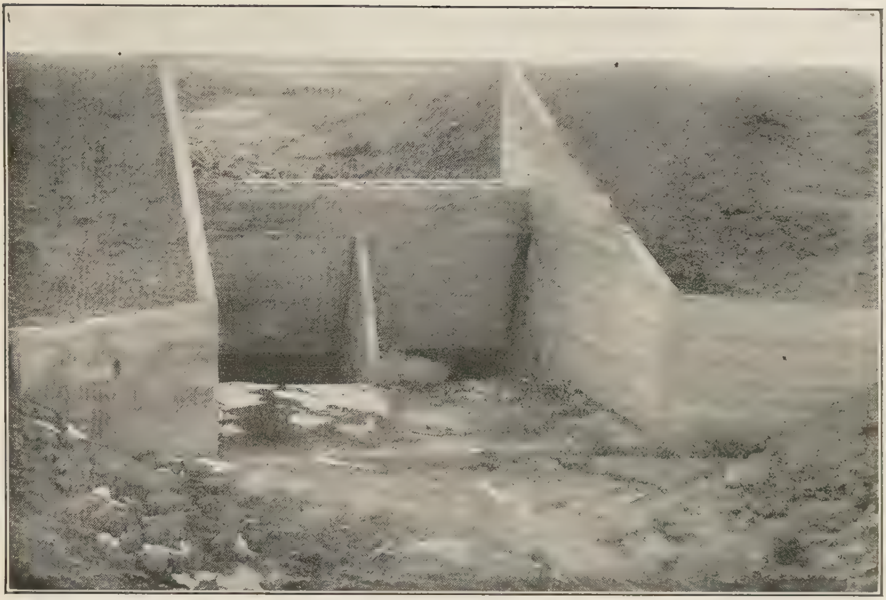

FIG. A.—Drop with baffle wall. Comanche Canal, Arkansas Valley. Sugar Beet & Irrigated Land Co., Colo.

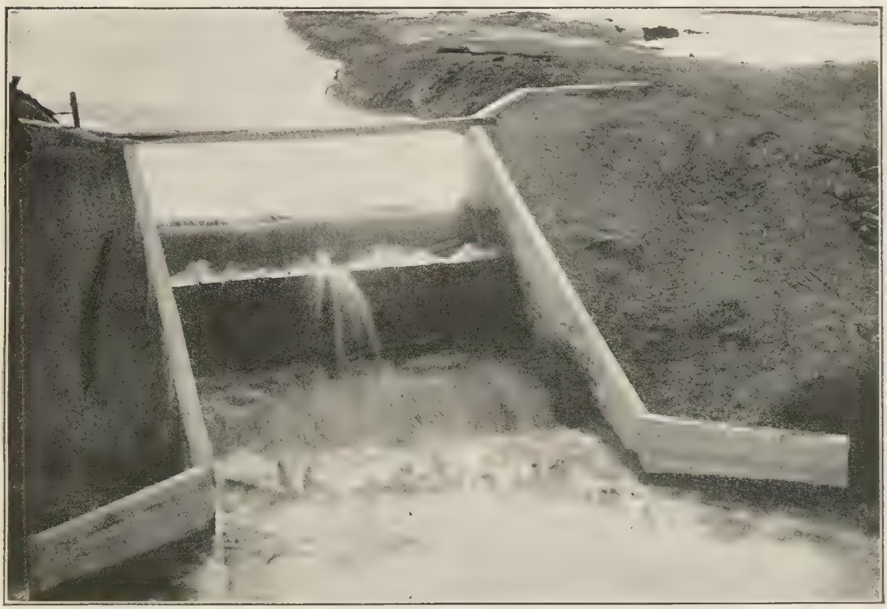

FIG. B.—Drop with baffle wall (same as above).

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PLATE XIII.

FIG. C.—Notch drop used by U. S. Indian Reclamation Service.

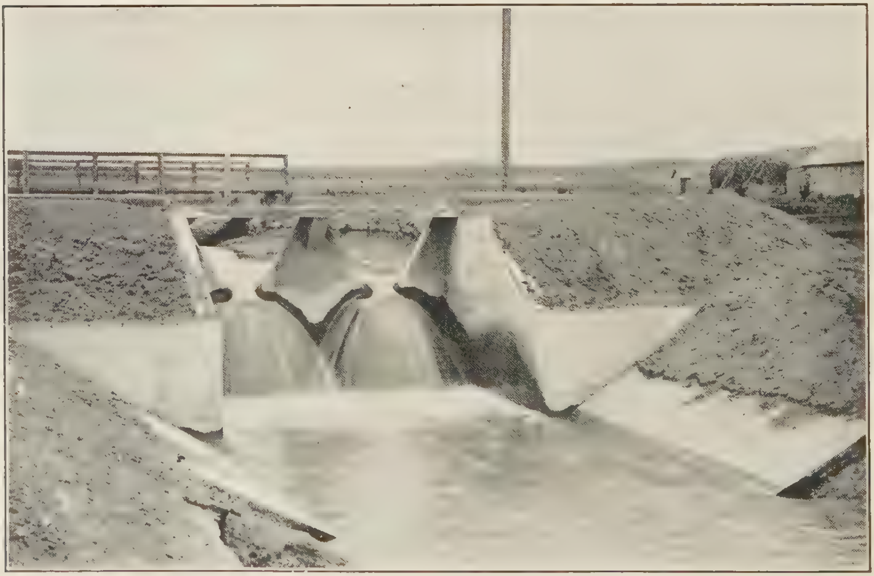

FIG. D.—Notch drop on the Mora Canal. Boise Project, Idaho.

wing walls are 9 inches thick at the bottom, taper to 5 inches at the top, have a footing 2 feet 9 inches wide. The side walls are 10 inches thick at the bottom at the maximum height, and taper to 5 inches at the top; they are braced by the baffle wall and divided into panels by vertical beams formed as part of the wall

COST OF DROP ON COMANCHE CANAL, COLORADO

Classification	Quantity, cubic yards	Unit cost	Total cost	
Excavation:				
Building dikes and ditches for controlling water during construction	....	....	$72.25	
Tearing out old wooden structure:				
Cost.................. 34.50				
Less material sold.... 25.00	....	......	9.50	
Rock excavation................	30	$0.752	22.56	
Earth excavation...............	253	0.49	123.89	$228.20
Concrete:				
Cement at R. R. station..........	....	2.735	246.18	
Hauling cement 4 miles..........	....	0.315	28.34	
Hauling sand and gravel.........	....	1.043	93.87	
Lumber delivered at job.........	....	2.186	196.71	
Building forms..................	....	4.892	440.25	
Nails and wire in forms..........	....	0.181	16.28	
Reinforcing steel at railroad......	....	3.268	294.11	
Hauling steel 4 miles............	....	0.302	27.15	
Cutting, bending and placing steel	....	2.005	180.43	
Mixing and placing concrete.....	....	2.417	217.52	
Protecting concrete from freezing	....	0.730	65.71	
	90	20.073	1,806.55	1,806.55
(Cost of material $711.48 = 7.905 per cubic yard				
(Cost of labor $1,095.07 = $12.167 per cubic yard				
Backfilling and puddling...........	540	0.445		240.36
Equipment:				
Use of tools and equipment......	....	146.96		
Camp expenses including hauling and setting up a camp outfit.	....	153.93		300.89
Engineering..........................	....	....		229.88
Design and supervision of construction	....	....		
				$2,805.88

and reinforced for cantilever action to support the ends of the panels.

The floor of the water cushion is formed of slabs arched on the under side, reinforced near both faces with ⅜- and ½-inch bars, spaced 8 inches apart in both directions. The downstream floor is 5 inches thick and supported on a substructure of reinforced beams resting on reinforced concrete posts. This form of floor construction is not necessary where an average firm soil foundation is obtainable.

This structure is more elaborate than necessary. The baffle wall is not necessary. The depth of the water cushion is larger than that commonly used for drops of this size without a baffle wall. Without a baffle wall the sloping upper part of the breast wall is objectionable, as it would discharge the water farther out in the cushion. With the breast wall made vertical, the water cushion, 17 feet long, would be of sufficient length.

The itemized cost of construction is given above and may be of value in preparing estimates of cost of similar comparatively elaborate structures built up of thin reinforced walls. The cost of the reinforced concrete in place was $20.073 per cubic yard, of which $12.65 represents the cost of the steel reinforcement in place and form work. The total cost of the completed structure is $2805.88 or about $31 per cubic yard of concrete.

Small Notch Drop on Huntley Project, Montana (Fig. 100).—This structure is a good example of a well-proportioned drop, conforming in general with the principles of design previously given. It is designed for a capacity of only 40 second-feet with a velocity in the canal of 1.22 feet per second. The concrete lined inlet section is longer than necessary for a drop of this size, especially with as small a velocity of approach as obtained in this case.

Notch Drops, North Platte Project, Nebraska-Wyoming.—On this project several notch drops of the general type shown in Fig. 101 have been constructed. This special design differs from the others in that the structure is built to be used also as a check gate and is combined with a lateral headgate. The carrying capacity of the canal is 136 cubic feet per second. The notch opening is formed with grooves on the sides for the insertion of flashboard to regulate the upstream water level when used as a checkgate. The lateral headgate discharges into a short con-

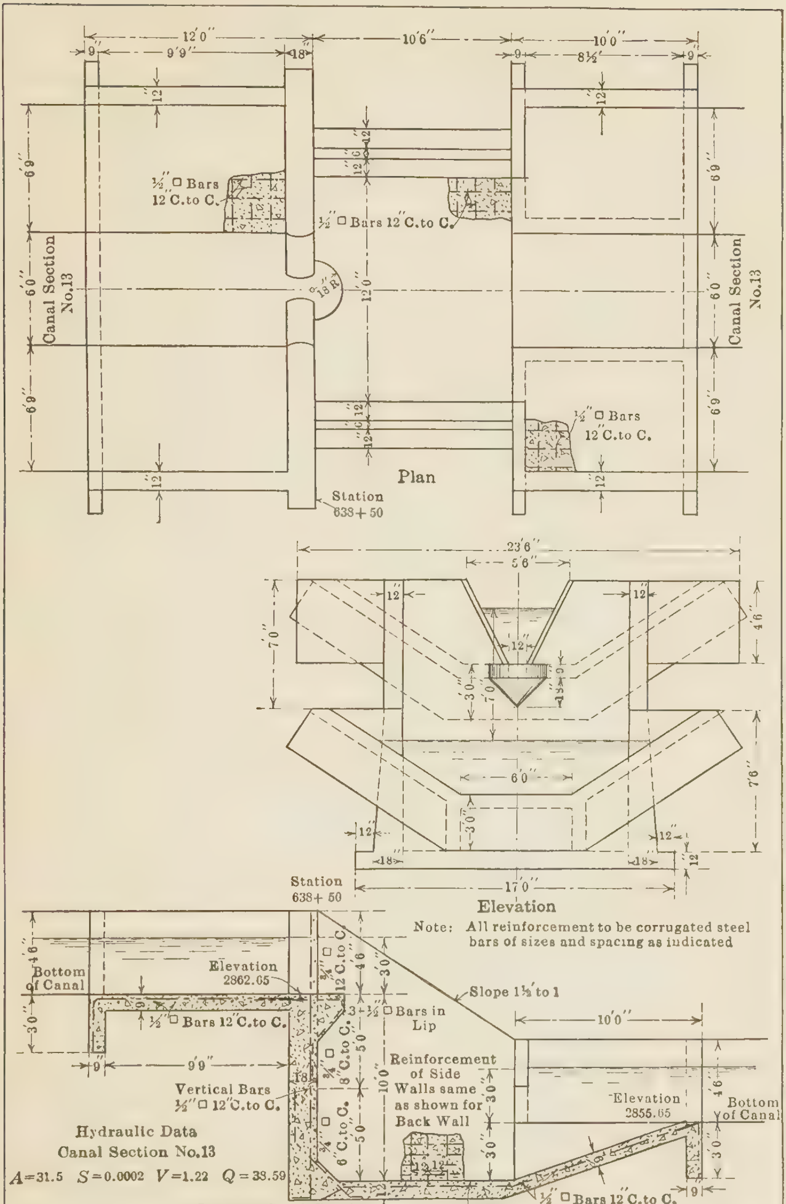

FIG. 100.—Small reinforced concrete notch drop. Huntley Project, Mont.

crete lined section, in which are placed five vertical concrete posts to act as baffles and check the velocity of the water delivered through the gate. The use of a notch drop where, as in this case, it is to serve also as a check gate is not usually desirable, because the checking of the water by raising the base of the notch

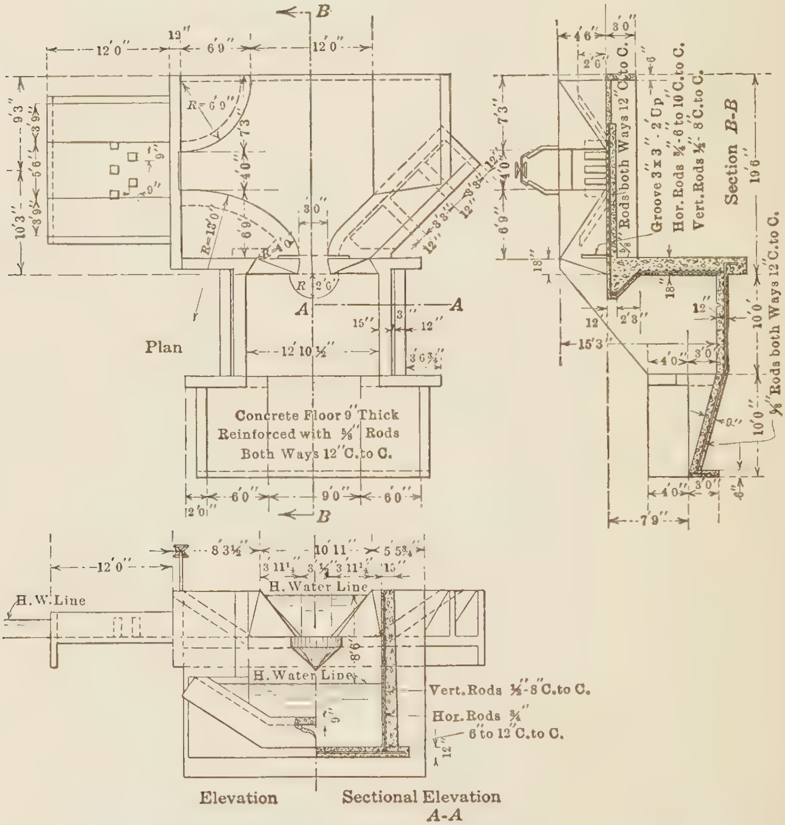

FIG. 101.—Notch drop and lateral headgate. North Platte Project, Neb.

with flashboards interferes with the correct action of the notch. The addition of baffle posts to check the velocity of discharge through the gate is not usually considered necessary.

The total cost and unit cost of five notch drops, containing a total of 323 cubic yards, is given below:

Cost of 5 Notch Drops, North Platte Project, Nebraska, Wyoming

Item	Total cost	Cost per cubic yard
Excavation and backfill	$282.47	....
Material haul	$152.96	$0.474
Moving equipment	31.79	0.098
Carpentry	289.73	0.897
Sand	226.97	0.703
Gravel	540.67	1.674
Water	71.14	0.220
Cement	702.70	2.176
Reinforcement	534.43	1.655
Concreting (mixing and placing)	395.01	1.223
Lumber	258.00	0.799
Lining banks	57.94	....
Miscellaneous	969.31	3.001
	$4,513.12	$12.920

Carpenters were paid $0.45 an hour and laborers $0.30 an hour.

Notch Drop on the Mora Canal, Boise Project, Idaho (Fig. 102 and Plate XIII, Fig. D).—Several drops of this form have been constructed on the Boise project. The design is very similar to that used on the North Platte project. The main points of difference are the form of the edges of the notch openings, the form of the water cushion, and the upstream floor and wings. The notch openings for the drop on the Mora Canal are formed in the extension of the breast wall above the canal bed; this wall is 9 inches thick and tied to the upstream buttress; the edges of the walls forming the trapezoidal notches are curved. The edges of the notches on the drop of the North Platte project have a special shape, probably intended to conform more nearly to the form used in India where the structures are built of masonry instead of reinforced concrete; there is no apparent necessity for this special shape and it increases the cost over that of a plain wall.

The Mora Canal drop is provided with grooves on the upstream side of the notches for the insertion of flashboards. This is open to the objections previously stated of using a notch drop for a check gate. The foot walk necessary for the operation of the flashboard is placed on the line of the crest of the breast wall, supported on the side walls at the two ends and on the inter-

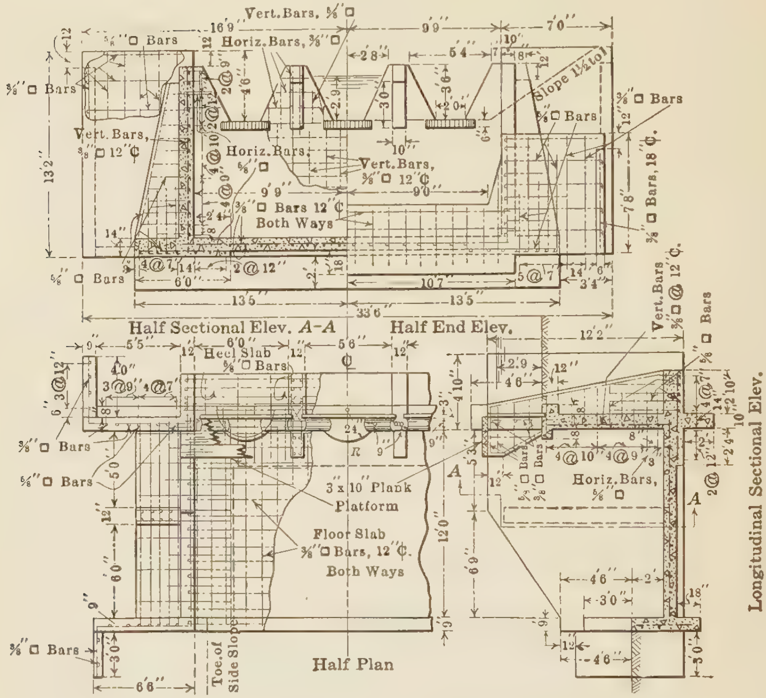

FIG. 102.—Reinforced concrete notch drop. Mora Canal, Boise Project, Idaho.

COST OF REINFORCED CONCRETE NOTCH DROP ON MORA CANAL, BOISE PROJECT, IDAHO

Item	Amount	Unit cost	Cost	Unit cost per cubic yard of concrete
Excavation....................	80 cubic yards	$0.80	$63.88	....
Backfilling and puddling........	150 cubic yards	0.57	85.88	$3.19
Concrete......................	47 cubic yards	....	....	....
Cement delivered............	275 sacks	0.75	204.25	4.35
Forms, labor.................	....	....	214.40	4.56
Lumber...................	2,612 feet	22 M.B.M.	90.54	1.93
Nails and wire	....	....	12.78	0.28
Mixing and placing..........	....	....	231.28	4.90
Miscellaneous...............	....	....	18.89	0.41
Reinforcing steel............	1,444 feet of ⅜-inch	0.02	28.88	0.62
	2,263 feet of ⅝-inch	0.04	90.52	1.92
Hauling and placing.......	....	....	24.54	0.52
				$22.68

Sand and gravel were hauled ⅜ mile, water 2 miles, other materials 6 miles. Wages for an 8-hour day were: Common laborers, $2.25; carpenters, $3.20; sub-foreman, $3.50; foreman, $4.00

mediate brackets built with the piers separating the notches. The absence of an upstream floor may be objectionable where the material is easily eroded. The form of water cushion used for the North Platte drop is considered preferable. Protection of the canal bed and sides is necessary at the outlet to the drop; this is not indicated in the drawing, but was constructed in the form of a concrete lining.

Four of these drops, two for a fall of 5.0 feet and two for a fall of 5.5 feet, were constructed on this canal. The cost of one 5-foot fall was as in foregoing table.

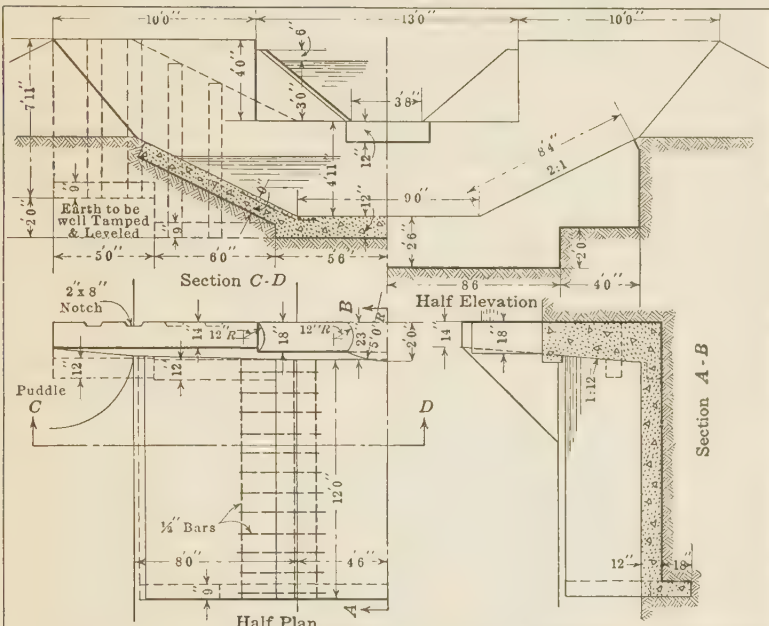

FIG. 103.—Notch drop. U. S. Indian Service.

Notch Drop for U. S. Indian Reclamation Service (Fig. 103 and Plate XIII, Fig. C).—This notch drop design is reduced to its simplest form. The drop consists of two parts: of the breast wall extending on both sides well into the canal banks to act as cut-off wings and the lined section of canal on the downstream side, terminating with a toe cut-off wall. In the breast wall is formed the one notch opening for a discharge capacity of 118 cubic feet per second; the shelf or lip at the base of the

notch does not project as far downstream as is usually considered desirable. The vertical grooves on the upstream face of the breast wall are probably intended to increase the resistance against the percolation of the water along this face. The breast wall is not reinforced. There is no water cushion, the water falls on the concrete floor 12 inches thick and the side slopes of

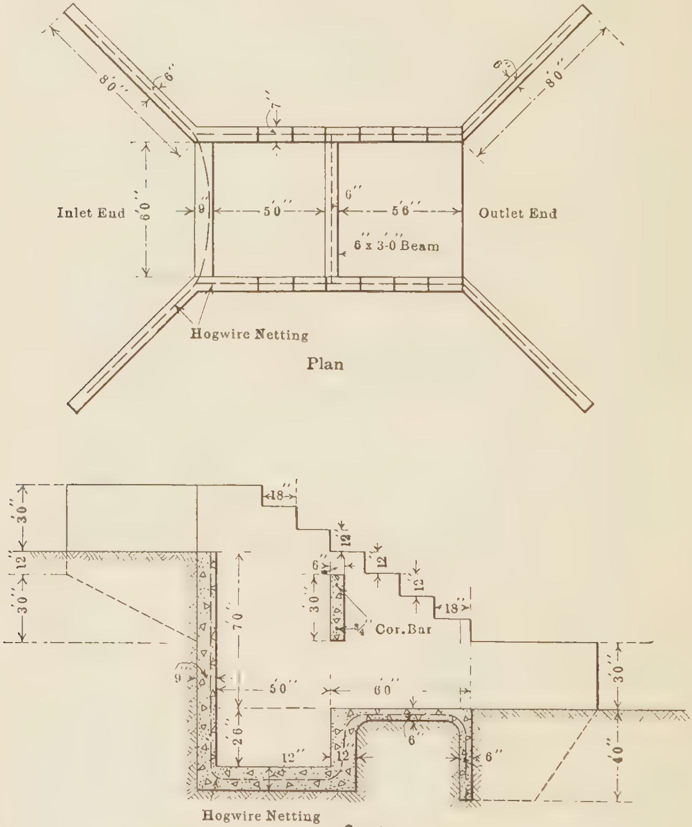

FIG. 104.—Drop on Carlton lateral. American Beet Sugar Co., Colo.

the canal are protected by a lining 9 inches thick, forming a concrete lined section 12 feet in length.

The use of a two-notch opening instead of a one would probably be preferable for this size of canal to divide the flow and decrease the force of impact of the falling water. The absence of a water cushion for a drop of this height and volume of water is

not advisable. A shallow water cushion could have been formed at a very small additional cost by depressing the end of the concrete lined section next to the breast wall to a depth of about 1½ feet below the canal bed.

Small Reinforced Concrete Drops on Carlton Lateral, American Beet Sugar Co., Colorado (Fig. 104).—On this lateral seven drops were constructed varying in height of fall between 2¾ and 7 feet. The drawing is for a 7-foot drop. The same design and dimensions were used for all sizes except in the height of side walls and in the concrete beam extending across the drop which was used only for the drops 7 and 6½ feet high. This beam acts as a strut to brace the side walls. The drops are reinforced with a central web of hog wire fencing, well lapped or wired together at all joints and corners. The length of the water cushion is too short for heights of fall greater than 4 feet; it should be about 10 feet for the 7-foot fall and would be best formed by substituting for the upper floor a sloping floor of the same thickness, joining the floor of the water cushion with the canal bed.

Cost of Drops on Carlton Lateral

Item	Height of fall				Average unit cost per cubic yard
	2¾ feet	4 feet	6½ feet	7 feet	
Amount of concrete, cubic yards	10	10½	11¾	12¼	
Labor:					
Excavating, men and teams	$8.75	$15.85	$14.24	$30.72	$1.563
Carpenter work on forms	10.95	8.40	12.55	14.50	1.043
Screening gravel	5.04	5.77	7.12	7.61	0.574
Hauling gravel	12.97	14.87	18.35	19.61	1.478
Mixing and putting concrete	15.75	13.80	23.87	18.90	1.625
Miscellaneous	6.95	7.96	9.83	10.51	0.792
	$60.41	$66.65	$85.97	$101.85	$7.075
Material:					
Cement	41 bags, $15.04	47 bags, $17.24	58 bags, $21.28	62 bags, $22.74	$1.715
Lumber	3.82	4.38	5.41	5.78	0.436
Nails	0.30	0.35	0.45	0.45	0.035
Wire netting	1.00	1.14	1.41	1.51	0.113
Cement for patching and plastering	0.14	....	0.14	0.15	0.011
	20.30	23.11	28.69	30.63	2.310
Engineering	20.55	23.56	29.08	31.08	2.343
Total cost	$101.26	$113.32	$143.74	$163.56	$11.728
Cost per cubic yard	10.12	10.78	12.25	13.35	....

The depth of the water cushion need not be greater than 2 feet for the larger height of fall.

The itemized cost of construction for three sizes of drop is given in the table above. The concrete mixture was one part of cement to six of sand and gravel. The cement cost about $1.46 a barrel and the gravel, screened and hauled, about $2.10 a cubic yard.

Wooden Standard Drop for Laterals on Boise Project, Idaho, Type No. 2 (Fig. 105).—The design is drawn for a height of drop of 4 feet, for a depth of water of 2 feet, side slopes of 1½ to

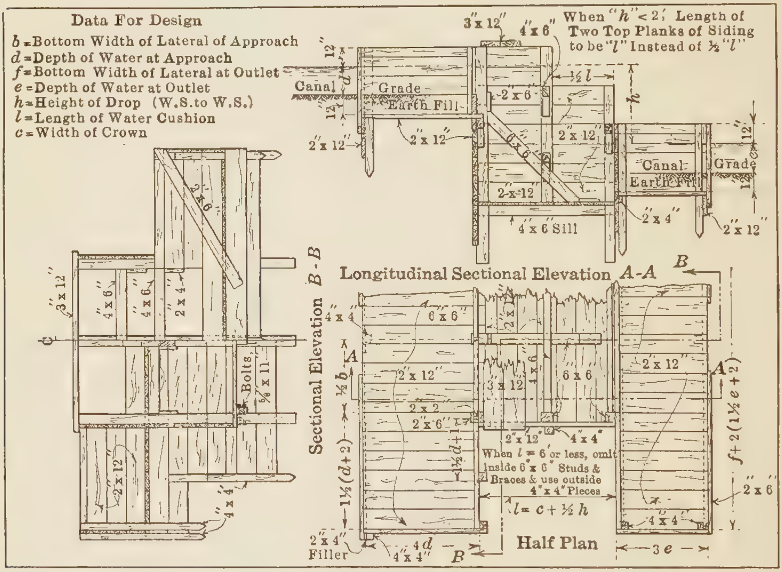

FIG. 105.—Standard wooden drop for laterals, Type No. 2. Payette-Boise Project, Idaho.

1 and a bottom width of canal of 8 feet. By modifications, some of which are indicated on the drawing, it is adapted to smaller or larger sized drops. The crest of the drop is designed for the insertion of flashboards to use the structure, also as a check gate. The length of the crest between side walls is equal to the bottom width of the canal, and in the drawing is divided into two bays by a built-up post with flashboard grooves. For a crest length of 6 feet or less this post is omitted. The inlet and outlet are formed by short flume sections, in which the ends of the canal are well boxed in. The total cost of 47 drops

and 44 checks on this project and the unit cost per 1,000 feet board measure is as follows:

	Total cost	Unit cost per M.B.M.
Hauling materials	$329.37	$5.60
Excavation and backfilling 4,047 cubic yards	1,905.36	32.40
Installing structures	785.55	13.37
Engineering (estimated)	176.33	3.00
Lumber 58,775 feet @ $20.25 M. B. M.	1,190.20	20.25
Nails and bolts	71.82	1.22
	$4,458.63	$75.84
Average cost per structure	49.00	

The above figures show that the cost of excavation and backfilling is a large part of the total cost.

Wooden Drop on Fargo Wasteway, Boise Project, Idaho.—On this wasteway were constructed two drops 5.5 feet high, one drop 6.0 feet high and one 7.0 feet high. The design is similar to that used for standard drops described above, modified for the greater height of drop and for the larger capacity of the canal. The drop is not used as a check gate. The breast wall is divided into three panels by vertical posts with diagonal braces. The length of the water cushion is 14 feet for 6.0- and 7.0-foot drops and 12 feet for the 5.5-foot drops.

The total cost of the four drops was as follows:

Hauling material and supplies	$23.63
Excavation and backfilling, 600 cubic yards	99.43
Making and installation	33.88
Engineering and superintendence	19.00
Lumber	123.74
Hardware	8.15
Miscellaneous	6.19
	$314.02
Average cost per drop	78.50

Assuming lumber to cost $20 per thousand feet board measure, as there are about 6,200 feet of lumber in the four drops, the completed cost of the drops will average about $50 per M.B.M.; this is much less than the cost for the smaller drops given above and represents more nearly the cost for wooden structures of this type under average conditions.

Wooden Drop on Corinne Canal, Utah Sugar Co., Utah (Fig. 106).—This structure is one of several drops of this type used on this system. The design illustrates a method of bracing the side and wing walls against earth pressure by anchor rods extending from the side posts to deadmen. This method is often practised for wooden drops or similar structures, in which the width between side walls is too great to be braced with struts extending between walls. The inlet and outlet wings and floor

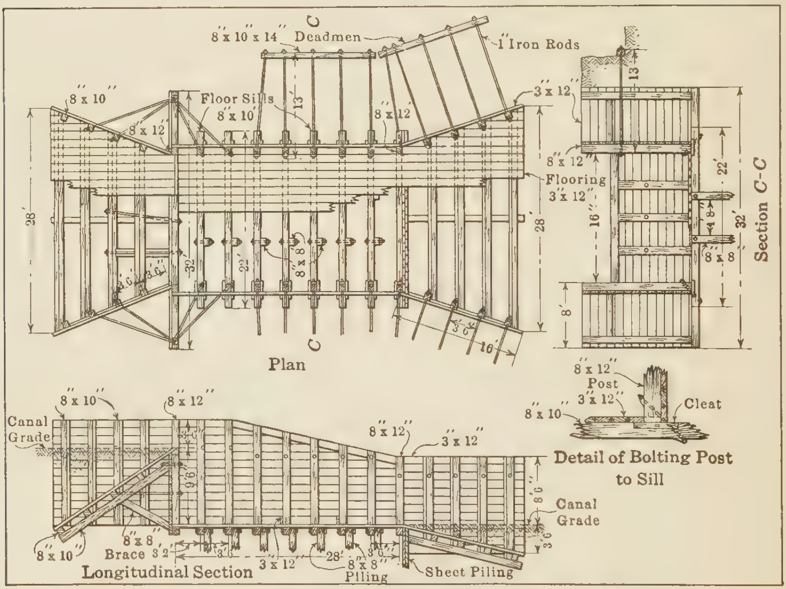

Fig. 106.—Wooden drop. Corinne Canal, Utah Sugar Co.

give a good secure connection with the earth bed and sides, such as is necessary with fine, loose soil easily eroded.

CHUTES OR RAPIDS

A chute is a lined canal, flume or pipe, placed on a steep grade, with an inlet and an outlet structure. A chute is used to take the place of a series of drops or as a substitute for a drop. It frequently forms the channel of a wasteway or escape.

Where the grade is not very steep a series of drops will usually be more economical as far as construction cost alone is con-

cerned; but there may be other factors to consider, such as the seepage loss, cost of maintenance, width of right-of-way which may justify the greater cost of a chute in the form of a concrete lined canal. Where the grade is very steep, as may be obtained for a canal located on the line of a steep ridge or along a steep wasteway, or where there is a sudden drop on the line of the canal, as in passing from a higher to a lower bench, or in delivering water down a steep incline from one canal into a canal at a lower elevation, the use of a chute will often be more economical than the use of one or more drops.

The limiting value of the grade beyond which it will be more economical to use a chute than a series of drops will vary largely with the volume of water, the form of design of the drops and of the chute. For example, with a canal carrying 500 cubic feet per second, with a maximum velocity of 3 feet per second, it was found that a chute 1,000 feet long formed as a canal lined with 6 inches of concrete, cost about the same as a series of reinforced concrete notch drops when the grade of the canal was about 2½ feet in 100 or 130 feet per mile. The economic height of drop for this grade and for the conditions assumed was found to be about 8 feet. With a short chute the inlet and outlet structures form a considerable part of the total cost and therefore make the comparison less favorable to the chute.

Flow of Water in Chutes Formed by Canal or Flume.—In a typical chute the water is delivered through an inlet structure and from this point the flow in the chute is accelerated down to a point where the increased velocity is sufficient to produce a frictional resistance which will prevent further acceleration. If the channel beyond this point is continued on the grade corresponding to this velocity and with a constant hydraulic radius the flow will be uniform with a constant velocity. Where this section of uniform flow is continued with a section of channel which has either a greater or smaller velocity, the change in velocities is provided by the use of a transition.

In a short chute on a steep slope the acceleration may continue down to its lower end without obtaining a velocity as high as that required to give a steady flow corresponding to the steep grade of the chute.

As far as the hydraulic computations are concerned, a chute may be considered as made up of transitions in which the accelerated flow or changes in velocities are obtained, and of short

lengths of channels in which the flow will be uniform. The design of a chute may require several trials in order to divide the total fall between the lengths of transitions and lengths of channel of uniform flow, so as to best fit the grades of the chute to the profile of the line on which it is constructed. Where the change in velocity is large and requires a long transition, it will usually be preferable to consider the total length as made up of shorter lengths and make the computations for each length. The greatest change in velocity is frequently at the upper end of the chute, in which case the inlet structure is often designed to produce a a relatively high discharge velocity.

The hydraulic computations for the transition have been previously explained (Vol. II, Chapter VI), the loss in head due to impact and eddies resulting from changes in cross section can be neglected if the changes are made very gradually, as is often the case. The computations for the flow of water in those sections of the chute in which there is uniform flow are made with the usual formulæ of flow in channels, except that with high velocities considerable air is absorbed by the water which increases its volume and requires a larger cross-sectional area. Measurements made by the Reclamation Service on several concrete rectangular chutes of the Boise project in Idaho show that as much as $\frac{1}{3}$ of the gross water area may be air. These measurements are interesting in that they show the adaptation of Kutter's Formula to velocities higher than those for which it is commonly used. The gross water area, including air, was obtained by direct measurements, and the net water area was obtained by computation from the measured discharge and direct measurements of the velocity. The velocity obtained by dividing the discharge by the gross water area gives a lower velocity than the measured velocity. The measured gross water area and the corresponding computed velocity will give one value of the coefficient of roughness "n" in Kutter's Formula, while the computed net water area and the measured velocity will give a smaller value of n. In making the computations for the design of a chute, the required gross water area will be obtained by using the larger value of "n," and the actual velocity will correspond to the smaller value of n. The measurements on the Boise project give the following results for the maximum discharges used.

EXPERIMENT ON CONCRETE CHUTES, BOISE PROJECT, IDAHO, TO DETERMINE N IN KUTTER'S FORMULA

I. Based on Measurements of Gross Water Area

Name of chute	Width in feet	Measured depth in feet	Discharge, cubic feet per second	Slope of water surface in feet per 1,000 feet	Computed mean velocity in feet per second	Coefficient of roughness, n
Mora wasteway....	5.0	0.32	27.50	81.00	17.69	0.0104
Valley mound.....	5.0	0.27	22.35	158.50	18.77	0.01119
Arena.............	6.0	0.315	50.40	210.00	26.67	0.01118
Lizard No. 1.......	3.06	0.36	16.42	82.44	15.05	0.0124
Lizard No. 2.......	3.08	0.306	16.42	198.50	17.66	0.0142

II. Based on Computed Net Water Area Computed from Measured Velocity

Name of chute	Width in feet	Measured depth in feet	Discharge, cubic feet per second	Slope of water surface in feet per 1,000 feet	Computed mean velocity in feet per second	Coefficient of roughness, n
Mora wasteway....	5.00	0.25	27.50	81.00	22.02	0.0078
Valley mound.....	5.00	0.17	22.35	158.50	26.34	0.0072
Arena.............	6.00	0.285	50.40	210.00	29.41	0.0098
Lizard No. 1.......	3.05	0.283	16.42	82.44	19.15	0.0091
Lizard No. 2.......	3.06	0.228	16.42	198.50	23.81	0.0098

These results are all for comparatively shallow depths. It is probable that for greater depths the difference between gross water area and net water area, which represents the amount of air in the water, would not be so large. The average value of the coefficient of roughness for the gross water area is about the same as that used for moderate velocities in smooth concrete lined canals. This value may be taken as about 0.012 for the computations of flow in chutes.

Measurements were made for depths of water much smaller than those given, and it was found that for very small depths wave action was very strong; the water would pile up at the crest of the wave, rush ahead and draw down the water so as to leave the chute nearly dry before the arrival of the next wave. For the larger depths used the wave action was very much decreased. This wave occurrence is probably due to the hydraulic phenomenon known as the hydraulic jump, the theory and extent of which is explained in Merriman's Treatise on Hydraulics. The formula given in this treatise for the height of jump probably gives excessive values, at least for larger depths of water; observations of a number of chutes would seem to indicate that a height of jump of 12 inches would seldom be exceeded.

The hydraulic computations and the adjustment of the sections

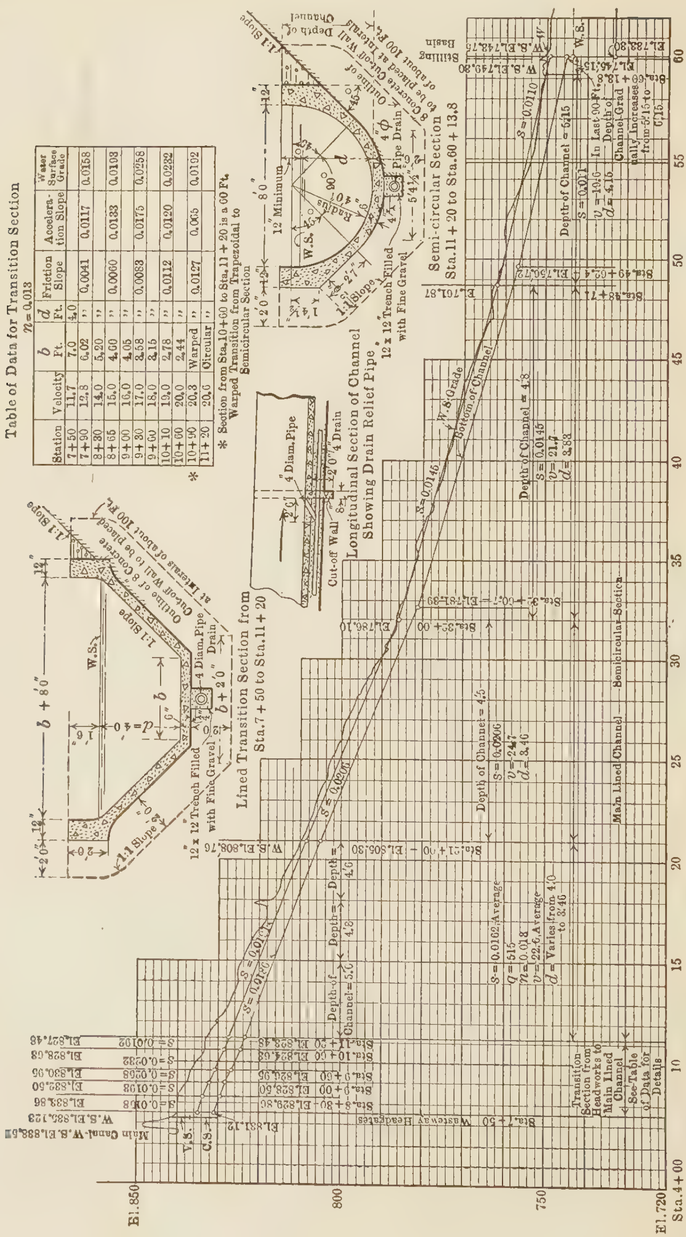

Table of Data for Transition Section
$n = 0.013$

Station	Velocity	b Ft.	d Ft.	Friction Slope	Accelera-tion Slope	Water Surface Grade
7+50	11.7	7.0	4.0			
7+90	12.8	6.02	"	0.0041	0.0117	0.0158
8+30	14.0	5.20	"			
8+65	15.0	4.60	"	0.0060	0.0133	0.0193
9+00	16.0	4.05	"			
9+30	17.0	3.58	"	0.0083	0.0175	0.0258
9+60	18.0	3.15	"			
10+10	19.0	2.78	"	0.0112	0.0120	0.0232
10+60	20.0	2.44	"			
* 10+90	20.3	Warped	"	0.0127	0.065	0.0192
11+20	20.6	Circular	"			

* Section from Sta. 10+60 to Sta. 11+20 is a 60 Ft. Warped Transition from Trapezoidal to Semicircular Section

FIG. 107.—Profile and cross sections of chute, Sulphur Creek wasteway. Sunnyside Project, Wash.

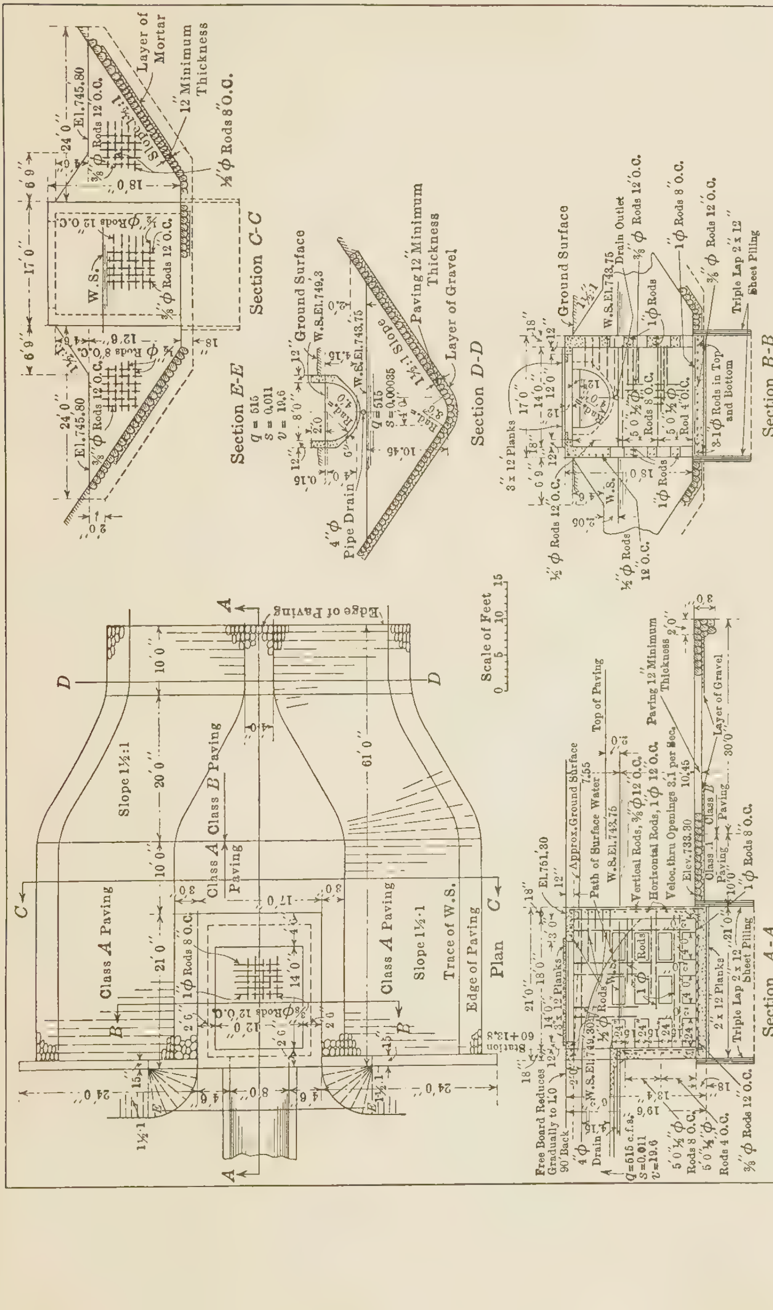

FIG. 108.—Outlet stilling box and basin on chute of Sulphur Creek wasteway. Sunnyside Project, Wash.

of a chute to the surface profile are illustrated by the chute designed for the Sulphur Creek Wasteway on the Yakima project, Washington (Fig. 107). The chute channel is formed of a concrete lined trapezoidal transition section for the upper 310 feet, connected by a warped transition section 50 feet long to the main part of the chute, which is a semicircular lined section 4893.8 feet long. The inlet to the transition is formed and regulated by headgates through which water is delivered with a velocity of 11.7 feet; the outlet structure at the lower end of the chute consists of a receiving box built in a stilling basin formed by a short section of enlarged canal. The receiving box is formed of a baffle end wall, opposite the end of the chute, and two side walls in which are a number of rectangular openings to diffuse the outgoing water (Fig. 108).

The transition is formed of short trapezoidal sections from 30 to 50 feet long, adjusted to the surface topography. The initial velocity at the upper end is 11.7 feet per second, and is accelerated to 20.6 feet at the lower end of the transition where it enters the main semicircular chute channel. In the transition a constant depth of water of 4 feet is maintained, and the cross section of the trapezoidal channel is decreased to conform with the increase in velocity by decreasing the width without changing the side slope. The accelerated flow in the transition is produced by dropping the water surface a height equal to the difference in velocity head plus a friction slope corresponding to the average velocity in the section considered. The hydraulic data for the transition section is tabulated on the accompanying drawing. For the computations it is divided into five sections, the last one of which is warped; each section is further divided into two sections by interpolation. To illustrate the computation, the first section from Station 7 + 50 to 8 + 30, 80 feet in length, has an inlet velocity of 11.7 feet per second and the water surface grade, which is parallel to the grade of the bed of the canal, is 0.0158. Since the friction slope corresponding to the average velocity of the section, if the section is short, will usually form only a comparatively small part of the total head, it can first be determined approximately by trial, and when subtracted from the total fall in the length of the section it gives the difference in velocity head. Assuming for the first trial an average velocity in the section of 13 feet per second, the water area is $\frac{515}{13} = 39.61$, the bottom width is 5.90, the hydraulic

radius is 2.30, and the friction slope by Kutter's Formula for a value of n equal to 0.013 is almost exactly 0.0041; this subtracted from the total desired grade of the water surface leaves 0.0117, which multiplied by the length of section, 80 feet, gives the difference in velocity head, or $\frac{x^2}{2g} - \frac{11.7^2}{2g} = 0.0117 \times 80$, from which x the velocity at the end of the section is equal to 14.0 feet per second. The variations in grade in the different parts of the main semicircular chute channel are small and produce small differences in velocities, which are provided for by keeping the same radius and varying the depth of the channel, which is extended above the horizontal diameter by vertical sides. The radius is 4 feet, the maximum depth of water is 4.15 feet, the freeboard is 1 foot down to the last 90 feet of the channel where it increases gradually from 1 to 2 feet at the lower end.

Flow of Water in Chutes Formed by a Closed Conduit or Pipe. —The hydraulic computations for a chute formed by a closed conduit are based in general on the same considerations of head lost in friction and of the difference in velocity head required to produce changes in velocity; with the difference that it is not necessary to give to the pipe the same grade as the grade corresponding to the hydraulic grade line. The computations are simplest when the topographic conditions make it possible to obtain, by the design of the inlet structure, an inlet velocity equal to the velocity corresponding to the hydraulic gradient of the pipe. For a high velocity this will require the use of a deep inlet well to obtain a head on the inlet of the pipe equal to the change in velocity head, plus the loss in entrance head. Where this head is not all obtainable from the design of the inlet, it may be necessary to join the inlet and the pipe chute with a short section of larger size pipe, in which the flow will be accelerated to the velocity required. These computations are well illustrated by the form of design used for several pipe chutes on the Sun River Slope Canal of the Sun River project in Montana. The hydraulic computations are given on the accompanying drawing of one of these chutes (Fig. 109, A and B).

The chute, designed for a maximum capacity of 890 cubic feet per second, consists of an inlet well, connected to the upper end of a pipe 8 feet in diameter and 194 feet long, continued by a transition 16 feet long, tapering gradually from 8 feet in diameter to 5 feet 6 inches at the lower end, where it joins to the

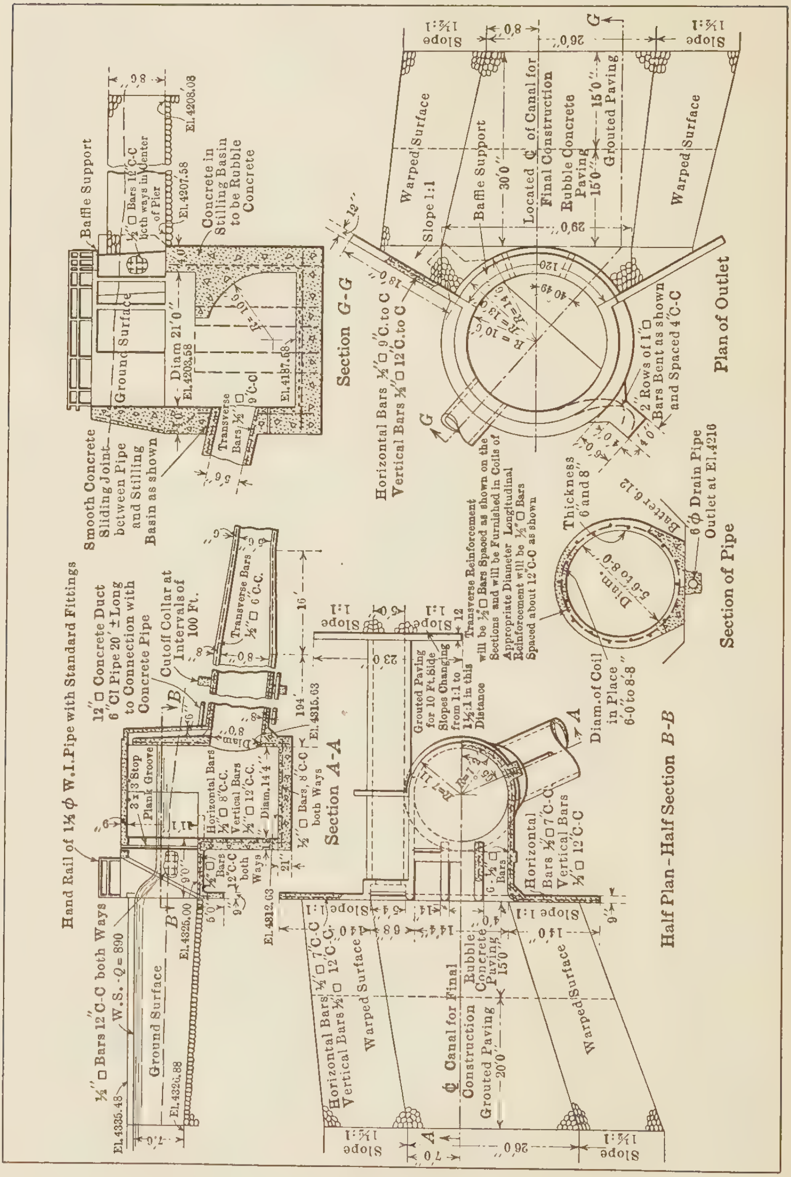

FIG. 109 A.—Pipe chute on Sun River Slope Canal. Sun River Project. Mont.

main pipe of the chute, 5 feet 6 inches in diameter, 1,126 feet long, discharging at the lower end into a stilling box. The velocity in the main pipe from Kutter's Formulæ for $n = 0.014$ and a fall of about 90 feet in the distance 1,126 feet is 37.5 per second. To obtain this velocity, the inlet structure was designed to produce a discharging velocity in the entrance end of the 8-foot pipe of 17.7 feet per second and from this point down to the junction with the main pipe the flow is accelerated up to the required velocity of 37.5 feet per second.

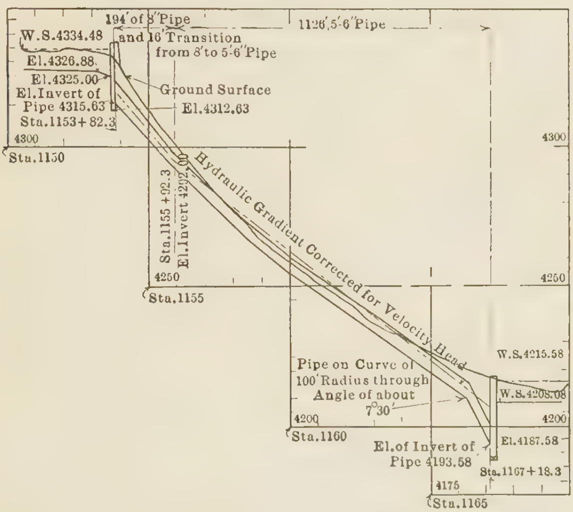

FIG. 109 B.—Profile of center line of chute. Sun River Project, Mont.

In detail the hydraulic computations are as follows:

First.—The water enters the intake well through the three openings, each 4 feet wide, with a depth of water of 6.35 feet, and an entrance velocity of 11.66. The drop in the water surface of 3.18 feet, is equal to the velocity head (h_v) of 2.12 plus the entrance head (h_e) equal to ½ of the velocity head or 1.06.

Second.—The water is discharged from the intake well into the head of the 8-foot pipe, with a velocity in the pipe of 17.7 feet per second. Because of the air duct connected to the upper end of the pipe, which permits air to enter, and of the accelerated flow below, the discharge into the pipe is the same as for a free orifice. Neglecting the velocity of approach in the inlet well,

which is comparatively small, the head required to produce the velocity 17.7 feet per second is obtained from the formula $Q = 0.62\,A\sqrt{2gh}$, and is equal to 13 feet, which is about the head used in the design.

Third.—The flow in the 8-foot pipe is accelerated from 17.7 feet per second to 37.5 feet per second. The hydraulic gradient required to carry 890 cubic feet per second in a pipe of this size if running full is 1 foot in 100, but the grade of the pipe and of the short section of transition is 11.25 feet in 100; this gives the excess fall which produces the accelerated flow. Below its upper end the 8-foot pipe will therefore run only partly full; the water area decreasing with the acceleration. The friction head in the 8-foot pipe is obtained with sufficient accuracy by using in Kutter's Formula the average velocity between the upper and lower end and the corresponding water area. This velocity is 27.6 feet and the corresponding water cross-sectional area is 32.3 square feet as compared to 50.26 when running full. The hydraulic radius is then 1.75 and for $n = 0.014$ the friction head is 3.5 feet in 100 or 7.35 feet in the total length of 210 feet. The velocity head corresponding to a change in velocity of 17.7 to 37.5 feet per second is 16.8 feet. The total head required is therefore 16.8 plus 7.35 or 24.15 feet; the actual difference in elevation provided in the invert of the pipe is 23.63 feet, but the fall obtained in the water surface is ample because it is greater than the fall in the invert by the difference in diameter of the pipes, of 2.5 feet.

Fourth.—The flow in the main pipe, 5 feet 6 inches in diameter and 1126 feet long, is uniform with a velocity of 37.5 feet per second. The corresponding hydraulic grade of Kutter's Formulæ for $n = 0.014$ is 7.9 feet per 100 feet.

Fifth.—The outlet is designed to destroy the high exit velocity but as this cannot be entirely obtained it is assumed that ¼ of the velocity head will be regained; this is equal to 5.5 feet.

The above example of the flow in pipe chutes differs from the more simple form of chute in that the hydraulic grade of the main section of the pipe is so steep that the velocity required in it to obtain uniform flow is greater than could be created in the intake structure itself, and therefore required a section of larger pipe in which there is accelerated flow. It is not necessary that this larger section be a pipe, for where the topographic conditions will permit it the accelerated flow may be obtained in an open

conduit connecting with a suitable transition to the inlet of the pipe chute.

Forms of Conduits for Chutes.—Open conduits for chutes are generally built as lined canals or flumes. Where the channel is in excavation it is usually built as trapezoidal or semicircular concrete lined canal with a minimum thickness of concrete of about 6 inches. Where the conduit is constructed on the surface of the ground, it is constructed as a bench flume, and where it must be built above the ground it is built as an elevated flume. Wooden flumes are usually not desirable for the leakage cannot be prevented and may result in severe erosion along and under the chute. Semicircular steel flumes have been used and are safer against leakage than wooden flumes. Concrete flumes supported on the ground have been used for a number of chutes on projects of the U. S. Reclamation Service. On account of the irregularities in flow, which occur to a greater extent with high velocities and small depth of water, a comparatively large freeboard should be provided specially toward the lower end of chutes. For velocities in excess of 15 feet per second, a minimum freeboard of 18 inches is considered desirable. To diminish the liability of wave occurrence and of conditions favorable to the hydraulic jump, moderately deep and narrow cross sections are better than shallow broad cross sections.

Closed conduits for chutes are usually formed as pipes and built of wood, reinforced concrete or steel. They may be preferable to open chutes for very high velocities, 20 feet or greater, on account of the irregularities of flow which may occur in the open chutes. Pipes can also be used to advantage where the profile of the ground surface on the line of the chute make it difficult or impossible to adjust open canals. Usually pipe chutes are more expensive than open channels.

Forms of Inlet Structure for Chutes (Plate XIV, Fig. A).—When the chutes form the escape channel of a wasteway, the inlet will usually be formed as the escape structure built to regulate the flow through the banks of the canal; it will include the waste gates on the downstream side of which sufficient fall is usually provided to give the flow at least a part of the acceleration which it must have in the chute channel.

When the chute channel is a portion of the canal itself, it may be necessary to form the inlet structure as a check gate to regulate the water level for upstream takeouts; or it may consist of a simple

transition section, the inlet of which may be formed as a trapezoidal notch to prevent the increased velocity upstream; this form is illustrated by the intake structure for the chute on the Strawberry Valley project, Utah, designed for a capacity of about 800 cubic feet per second (Fig. 110).

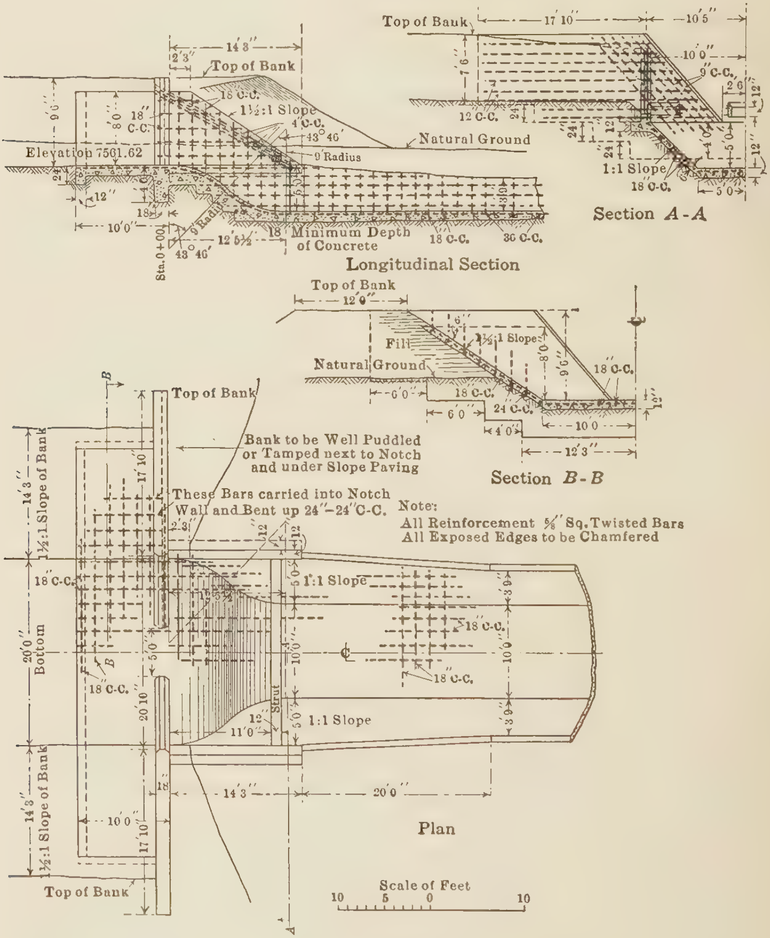

FIG. 110.—Inlet end of chute on Strawberry Valley Project, Utah.

The form of the intake structure will depend also on whether the chute channel is an open conduit or a pipe. The intake for a pipe chute will often be about the same as that used for an inverted siphon, and in its simplest form may be a conical tapering

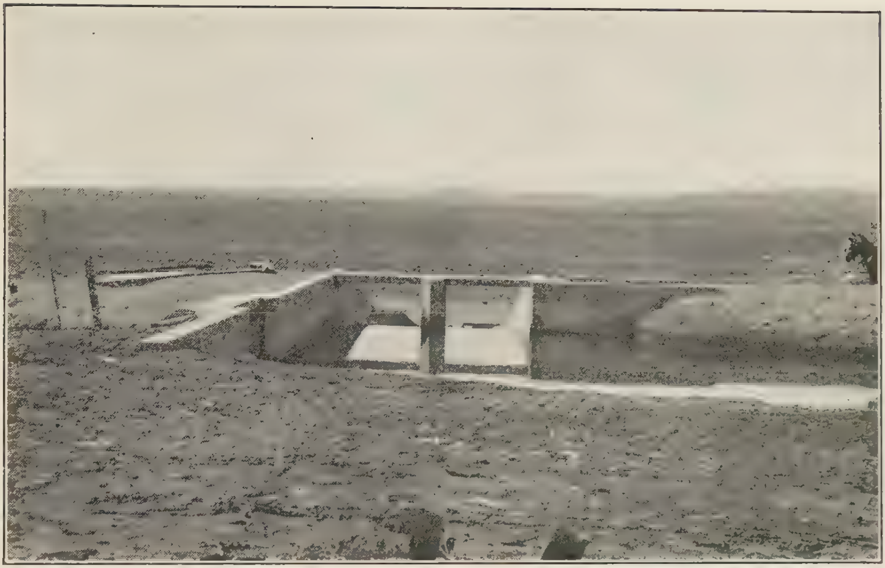

Fig. A.—Inlet end of chute with flashboard regulation. Umatilla Project, Ore.

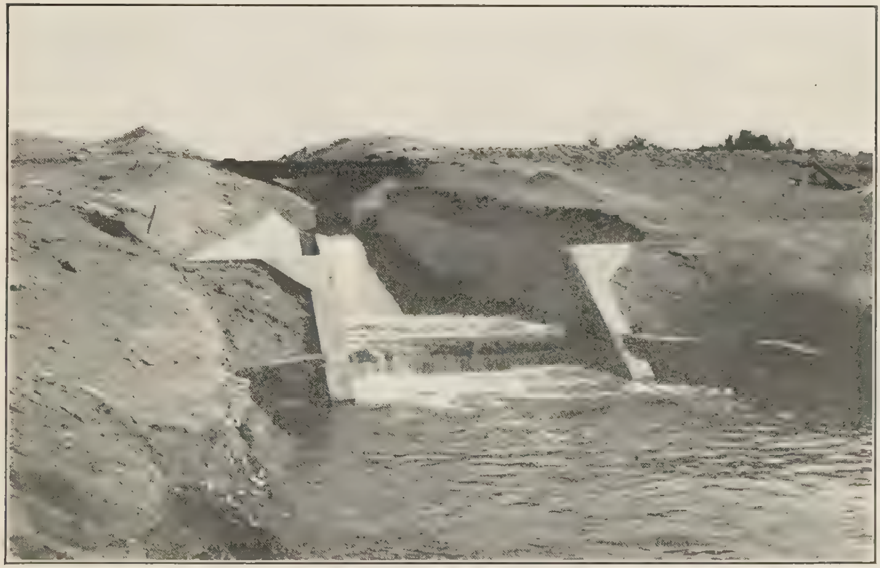

Fig. B.—Stilling basin at outlet end of chute. Umatilla Project, Ore.

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FIG. C.—Inlet to small pipe chute. Umatilla Project, Ore.

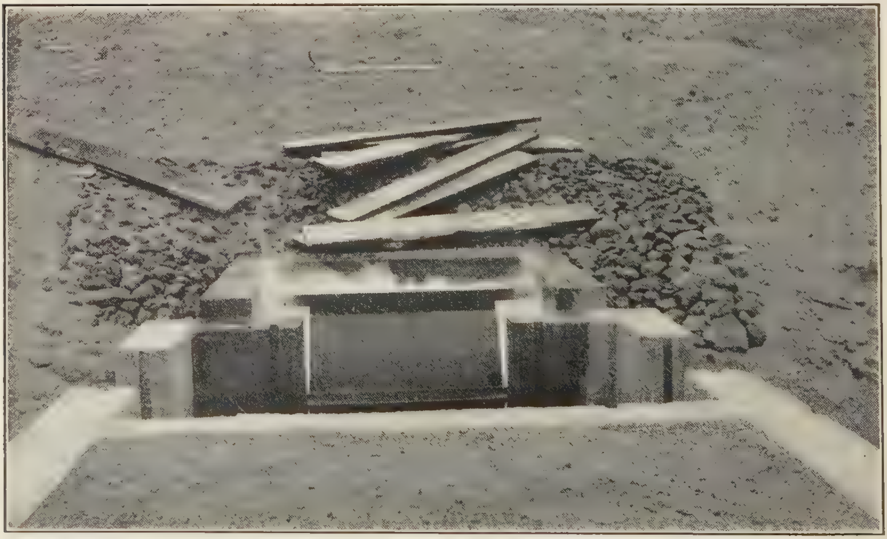


FIG. D.—Outlet to pipe chute with sheet metal curved baffle. Truckee-Carson Project, Nev.

inlet sufficiently deep to acquire a head equal to velocity head and entrance head corresponding to the velocity required in the pipe; as illustrated by the form used for small chutes on the Umatilla project, Oregon (Fig. 111 and Plate XIV, Fig. C). One form of intake structure for a large pipe chute is illustrated by that used on the Sun River project, Montana (Fig. 109).

Form of Outlet Structure.—There is a considerable diversity in the forms of outlet structures. The main object is to destroy the high exit velocity and resist the impact force. Two general types are used. In one type the water is discharged into a box, the jet of water striking baffle walls or baffle posts and issues out of this box, through openings or under baffles so as to be diffused before it passes in the canal. In the other type the water is

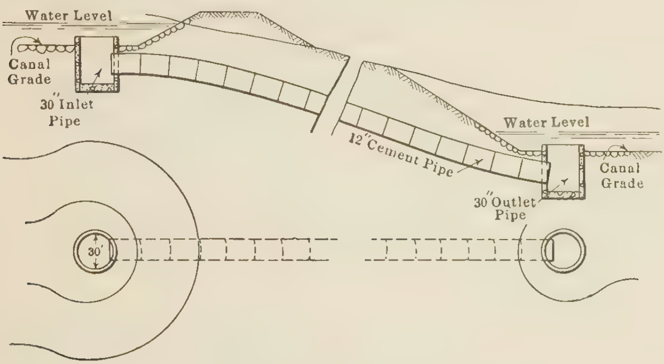


FIG. 111.—Typical pipe chute with 30-in. special pipes for inlet and outlet. Umatilla Project, Ore.

discharged directly in a large body of water or stilling basin, formed as an enlarged section of the canal usually with a cross weir wall at the lower end, over which the water passes before it enters the regular canal section. The two types are sometimes combined by building a receiving box or baffle posts inside of the stilling basin. The use of a stilling basin alone has the advantage of simplicity and if properly designed is probably the best form of outlet. This type of outlet is illustrated by those used on the Strawberry Valley project, Utah (Fig. 112) and on the Umatilla project, Oregon (Plate XIV, Fig. B). The stilling basin outlet of the chute on the Strawberry Valley project received a flow of about 800 cubic feet per second with a computed velocity of about 25 feet per second. The length of the chute

is about 1,700 feet. The stilling basin has an expanding section ending with an overpour weir wall which controls the depth of water in the basin. The crest of the wall is 60 feet in length, and about 2 feet above the floor of the basin. The depth of water

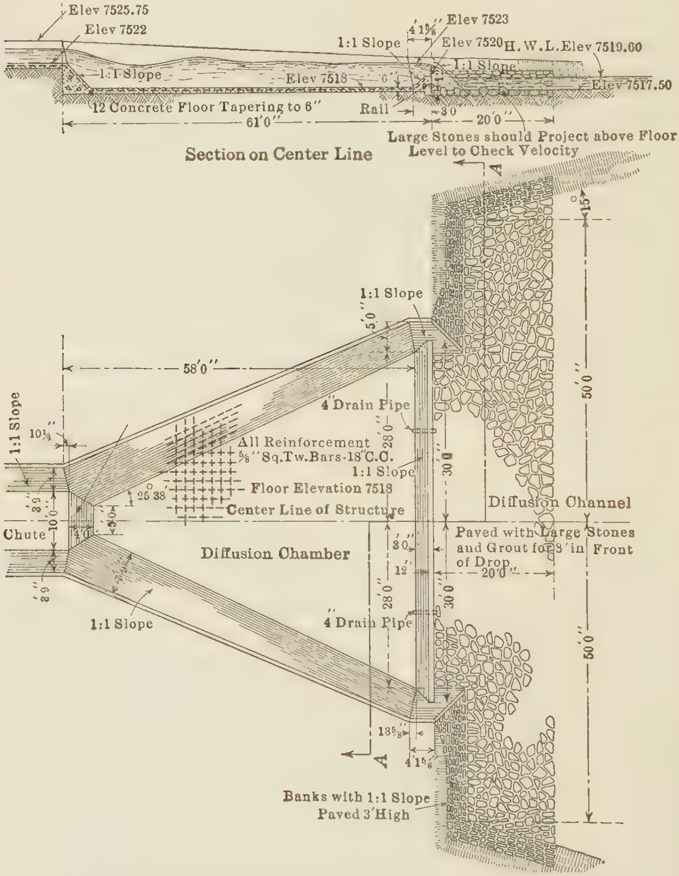


Fig. 112.—Stilling basin at outlet of chute on Strawberry Valley Project, Utah.

on the crest for about 800 second-feet flow is about 2.5 feet, which gives a depth of water cushion of 4.5 feet.

The stilling basin on the chute of the Umatilla project is essentially of the same type (Plate XIV, Fig. B).

The use of a receiving box with a vertical baffle wall placed transversely in the box against which the water is discharged,

will cause the water to splash upward and may require that the box be covered in order that the water be not splashed over the top of the box and wash away the soil around the structure. This action was very noticeable on chutes of this type built on the Boise project, Idaho, a form of which is shown in Fig. 113.

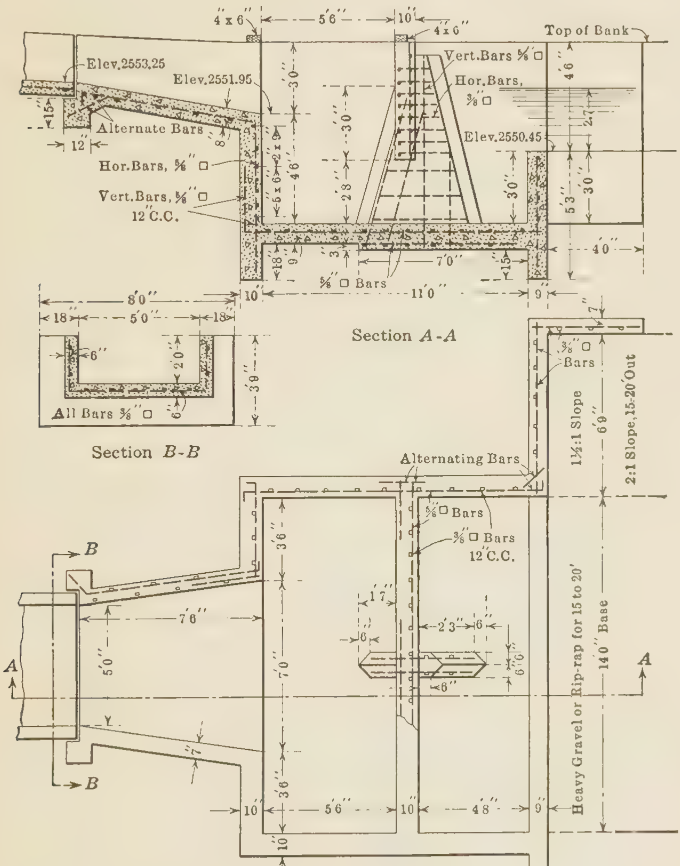


FIG. 113.—Receiving box with baffle wall at chute outlet. Boise Project, Idado.

This chute is rectangular in cross section, designed for a maximum capacity of 120 second-feet, drops 42 feet in a distance of 325 feet. The stilling box for this chute is shorter than on other chutes of this project, and is less effective in stilling the water. The shorter length was considered permissible in this case be-

cause the chute is a wasteway channel used infrequently and only for short periods. On stilling boxes of other chutes of about the same capacity the undershot baffle wall is placed in about the same position, but the stilling pool is 35 feet long, and 14 feet downstream from the baffle wall a raised sill or overpour wall 3 feet high is placed, so that the water discharged against the baffle wall is deflected back and flows under it, and then up over the overpour wall.

The same type of stilling basin with baffle wall is shown by that used for a chute on the Medina project, Texas (Fig. 114). The chute is a concrete lined trapezoidal channel designed for an outlet velocity of 30 feet per second and a discharge of 600 second-feet. At the lower end of the chute the cross section widens and in the center a concrete cutwater divides the flow

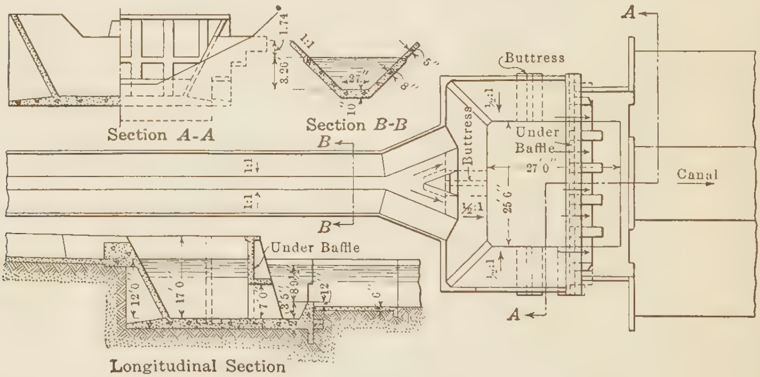


FIG. 114.—Stilling basin with baffle wall for chute. Medina Project, Texas.

and produces a more nearly equal distribution of the shock of the water on the undershot baffle, by spreading the flow on both sides. The structure and the outlet concrete lined section are reinforced.

The upward splashing of the water may be prevented by using in the place of a vertical baffle wall one sloping upstream or curved, so that the water will strike it on a more acute angle and deflect the flow downward. The curved form was given to the baffle wall of the pipe chute on the Sun River project, Montana (Fig. 109) and to a curved steel-plate baffle on the Truckee-Carson project, Nevada (Plate XIV, Fig. D).

A form of outlet which combines a large stilling basin with a baffle box is illustrated by that used for Sulphur Creek wasteway

chute, previously described (Fig. 108). This type of receiving box will destroy the high velocity and the 2 × 4-foot outlet openings in the side walls will diffuse the flow uniformly in the basin, but may be subject to obstruction by brush sticks or large material carried by the water. In this case the chute is a waste-way channel used only occasionally, so that the liability of obstruction would not be as serious as when used continuously. The use of a baffle wall inside of a large stilling basin, so that the water splashing will fall inside the stilling basin, is a good combination.

For a pipe chute, the outlet end of the pipe should be placed at the bottom of the discharge box or funnel-shaped basin, in order that the high velocity will be destroyed by or diffused in the larger volume of water above the outlet. This form of outlet for small pipe chutes is illustrated by that used on the Umatilla project, Oregon (Fig. 111). For large pipe chutes the design used on the Sun River project is interesting. In this outlet structure the water strikes the curved baffle of the circular well, which deflects the flow downward, and the water then rises and escapes through three openings above the baffle, each about 7 feet 4 inches wide, with provision for the insertion of baffle bars made of $3\frac{1}{2} \times 3\frac{1}{2}$ T-bars, spaced about 12 inches apart. These bars will help in stilling the water by diffusing the outgoing flow, but will be objectionable if the water carries large material which may cause obstruction.

REFERENCES FOR CHAPTER VII

References on Drops:

Maconchy, G. C.—Notch Falls—Part of Memorandum on the Different Methods of Ascertaining the Discharges of Rivers, Canals and Open Channels and on the Discharges of Orifices and Overfalls, etc.—Irrigation Department Government of Bengal, Calcutta, India.

Reid, A. G.—Notched Falls—Punjab Irrigation Branch Paper, No. 2—1894.

d'Rohan, W.—Some Methods and Costs of Irrigation Construction on the Rock Creek Conservation Co. Project at Rock River, Wyoming—Engineering Contracting—Dec. 27, 1911.

References on Chutes:

Murphy, D. W.—A Reinforced Concrete Spillway with Concentrated Crest Length—p. 278 Engineering News—Sept. 9, 1909.

Chutes on Standley Valley Irrigation System Near Denver—p. 462 Engineering Record—Oct. 23, 1909.

NIKOLITCH, M.—Concrete Chute in an Irrigation Canal—p. 557 Engineering Record—April 23, 1910.

HANNA, F. W.—Reinforced Concrete Chutes on the Boise Project—p. 542 Engineering Record—May 3, 1913.

Main Canal of Medina Irrigation Project—p. 424 Engineering Record—Oct. 18, 1913.

COLE, D. W.—Concrete Chute Drops Water 130 Feet from Canal to Reservoir—p. 456 Engineering Record—April 10, 1915.

CHAPTER VIII

DISTRIBUTION SYSTEM

Parts of Distribution System.—The distribution system consists essentially of the channels required for the conveyance of the water to all parts of the irrigable area and of the structures required for the regulation, division and delivery of water.

The channels of the distribution system (Fig. 115) include:

1. The main canal commanding the entire irrigable area and supplying the main laterals.

2. The main laterals commanding the main divisions of the irrigable area and supplying sub-laterals and distributaries.

3. The sub-laterals used when a main lateral subdivides into two or more branches.

4. The distributaries which are supplied from the main laterals or sub-laterals and convey the water to the farm units.

5. Natural channels or sloughs favorably located, used on a few systems for the conveyance of irrigation water.

6. The waste or tail channels, which are the extensions of the channels of the distribution system, down to a natural water course, depression or other point where the waste or unused water can be disposed of.

The structures required for the regulation, division and delivery of water are:

1. Check gates to control or raise the water level in the supply canal or lateral, in order to divert the required flow through one or more lateral headgates or delivery gates upstream from it.

2. Lateral headgates at the head of laterals or distributaries and delivery gates at points of delivery to farm units.

3. Measuring boxes or devices for the measurement of water at points of delivery to farm units, and in some cases at the head of laterals and distributaries.

Other structures required are those for the protection of the system and miscellaneous structures. Those required for the protection of the system include: wasteways and escapes to dispose of excess water or to discharge the entire flow in a waste

channel; drops, rapids or chutes to absorb excess grade and prevent excessive velocities, and, in a few cases, sand boxes or sluices

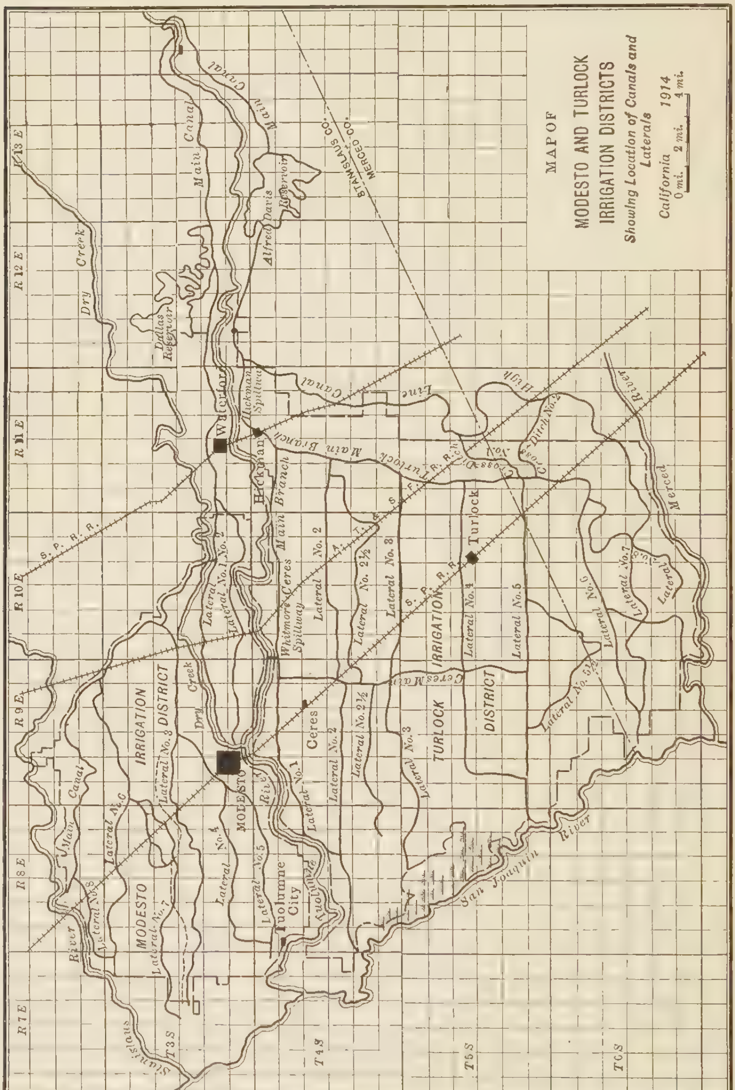


Fig. 115.

to prevent or remove deposits of material transported by the water. Miscellaneous structures are: (1) those required at the crossing of drainage channels or depressions, which include

flumes, inverted siphons, culverts, surface inlet or level crossings, and overchutes; (2) those required at road crossings which include bridges and culverts. Wasteways, escapes, sand boxes or sluices, flumes, inverted siphons and drainage crossings form usually a more important part of the diversion canal and have been considered in previous chapters and in Vol. I; drops, chutes, or rapids, while they may be of more frequent occurrence on laterals than on the diversion canal, have also been discussed in a preceding chapter in connection with the larger structures used on the diversion canal.

The structures required for the operation of the distribution system are discussed in following chapters.

Relation of Main Canal to Laterals and Distributaries and to Drainage Channels.—The watersheds of many streams will usually contain large bodies of irrigable land on both sides of the stream. When the available stream flow is sufficient to serve all the irrigable land, or where the stream flow can be used most favorably on selected parts of the irrigable land on both sides of the stream, the irrigation project will then consist of two separate systems, with separate diversion canals both diverting water from the same stream, usually at the same point, by means of a common diversion weir.

The diversion canal conveys the water from the point of diversion to the highest point of the area to be irrigated, which is the head of the main canal. The main canal usually commands all the land to be irrigated, from the main canal down to the main drainage channel or stream. The area commanded seldom has a uniform slope, but is usually divided into basins separated by ridges. Along the lower part of each basin is a more or less well-defined natural drainage channel or a continuation of depressions. The main canal is generally located on a flat grade along the higher boundary of the irrigated land; it intersects the ridges at points which are the locations of the head of the main laterals, is continued to serve all the main laterals, and usually ends as one of the laterals. The main laterals are located as nearly as possible on the line of the ridges, each one commanding the land on both sides from the watershed or ridge line down to the drainage line of each basin. Where a basin is divided into sub-basins by secondary ridges, sub-laterals heading at the main lateral will run down the secondary ridges. The slope along these ridges will in some cases be steeper than can be given to the grade of

the laterals and may require the use of numerous drops or chutes. The main laterals and sub-laterals are the source of supply for the distributaries which convey the water to the farms situated on both sides of the ridges.

To obtain better regulation of the flow in the distribution system it is desirable to eliminate unnecessary variations in flow in the main canal and main laterals. Therefore deliveries to farms should be made directly by the distributaries, and as far as possible no deliveries should be made directly from the main canal and main laterals. The general course of the distributaries is from the ridge or lateral down toward the drainage channel. Where the slope of the land is uniform, it is desirable that distributaries follow property lines or roadways, in order not to cut up farm units. In some cases this will require that a distributary run along two sides of a farm unit instead of diagonally across. If the farm unit is square, the length of the longer course along the two sides will be 1.4 times that of the shorter diagonal course, and will correspondingly decrease the available grade along the longer course. This will be an advantage if the diagonal shorter course gives an excess in fall, but otherwise will produce a smaller velocity, and therefore require a slightly larger cross section, which, with the greater length, will make the cost of earthwork for the longer course about 1.5 times that of the shorter course. On the other hand, the short diagonal course will cut up the farm unit in two or more parts, which will require: field bridges, two delivery gates to each farm unit, increasing the cost of the farm distribution system and making farming operations more difficult and expensive. A consideration of all the items of cost and of the disadvantages of the shorter course will usually make it preferable to locate the distributaries along the property lines, whenever the topography will permit it. The shorter diagonal course will be necessary when irregularities or ridges in the topography of the farm units fix the position of the distributary, or when it is necessary to reach certain high points which can only be reached by using as short a distributary as possible on a flat grade. On one project in California too strict adherence to the location of distributaries along property lines brought portions of them across low ground, where the canal section was largely or entirely in fill, and it was later found necessary to replace them by shorter diagonal courses which placed them along the high land where they belonged.

General Types of Distribution Systems for Land with Smooth Uniform Slopes.—Where the distribution system serves flat valley land or land which has a uniform unbroken slope from the main canal down to the lower boundary of the land to be irrigated, with no ridges to indicate the position of the laterals and sub-laterals, several plans for the location of the distribution system are feasible. Usually a few irregularities in the topography will indicate the best plan, but in some cases the best solution can only be obtained by the economic comparison of at least two general plans.

The first general plan will consist in placing the main lateral transverse to the main slope, on a flat falling contour grade skirting the upper boundary of the land to be irrigated; the laterals will then be run longitudinally with the slope down the steepest grade, supplying the distributaries which take out water from both sides of each lateral. The distributaries will then be across the slope, on a flat grade, serving farm units only on the downhill side. The flat grade of the distributaries is favorable to a comparatively large seepage loss, and will make it desirable to shorten their length by spacing the laterals comparatively close. By this plan the strip of land between two laterals is served by distributaries of about equal length running from each lateral toward the center of the strip of land but stopping at the last point of delivery. The length of the distributaries is therefore less than ½ of the average width of the strip. A spacing between laterals not greater than 2 to 4 miles, depending on the character of the soil, with corresponding lengths of the distributaries of about ½ to ¾ miles and 1½ to 1¾ miles, respectively, is desirable for methods of irrigations requiring large irrigating heads, such as 10 to 20 cubic feet per second, as commonly used in Arizona and Central California for the check flooding method of irrigation. A spacing between laterals not greater than 1 to 2 miles is desirable where smaller irrigating heads, of 2 to 4 second-feet, are delivered to farm units.

Where the average length of laterals is not great, such as when the width of the body of land, between the main canal and its lower boundary is comparatively small; then it may be more economical to eliminate the distributaries and place the laterals sufficiently close together to serve as distributaries. For farm units of 40 acres this will require laterals serving land on both sides, ½ mile apart. Each lateral will then carry a comparatively

small flow, which makes this plan specially desirable where the surface slope is so steep as to require a large number of drops or chutes for laterals of large capacities spaced farther apart.

The second general plan will have the main canal placed longitudinally with the slope, down the steepest grade, which may require numerous chutes or drops; the laterals will be placed transversely to the slope, on a flat, falling contour grade, and the distributaries will run down the steepest slope with deliveries to the farm units on both sides. The steep slope of the distributaries is favorable to a small seepage loss and permits the use of longer distributaries than in the previous plan. But in this second plan the distributaries extend only on one side of the supplying lateral. Therefore the laterals, if spaced the distances given above for the first plan, will give distributaries of about double the length, which because of the steep slope is not excessive. The distributaries will in general deliver to tiers of farm units on both sides and will therefore be spaced in general twice as far as in the first plan. The favorable position obtained for the distributaries will in many cases make it the most economic plan, both in first cost of construction and in the efficient conveyance of the water.

Importance of Distributaries.—On most projects the aggregate length of the distributaries will be several times that of the laterals and main canal, and they serve probably the most important function of the system in delivering the water to each farm unit; they therefore deserve careful consideration in the plans of the distribution system.

On many older projects the construction of the distribution system by the company or district has been confined to the main laterals; this has required the construction of distributaries and in some cases sub-laterals many miles in length by the individual water user or by an association of the water users. On other projects the main laterals, sub-laterals and a few main distributaries have been constructed to convey the water within a comparatively short distance of the farms, usually within 1 mile or less. On many of the new projects it has been found desirable to construct the complete system with all the distributaries required to deliver the water to the highest point in the boundary of each farm unit at an elevation sufficient to reach the highest irrigable land in the unit.

The result of experience shows that construction of any part

of the system by individuals or by water users is not desirable; it usually results in poor construction because of inexperience in this kind of work, in poor location, and in duplication of distributaries. It also places this additional labor and hardship on the farmers at a time when they have all they can attend to properly in the development of their farms. In the operation and maintenance of the system similar difficulties have resulted where the main canal and laterals are under the control of the management of the company or system, and the distributaries are operated and maintained by individual farmers or groups of farmers. The result has usually been poor maintenance, poor operation, considerable friction between farmers on the same distributary and dissatisfaction with the management of the company. This is especially well demonstrated by the experience on at least one project of the Reclamation Service. This project includes two separate systems having the same source of supply and serving two similar bodies of land, one on each side of the river. The land is similar and the two systems are under the same central management, but in one system the operation of the distributaries was left to the farmers, while in the other system the operation of every part of the system is under the same management. On this project general dissatisfaction resulted on the system where the distributaries were operated by the farmers, while on the other general satisfaction prevailed.

Natural Channels for the Conveyance of Irrigation Water.—Large natural channels, such as streams, creeks or sloughs, are occasionally so located with respect to the diversion canal or main canals that the flow from these canals may be discharged into them and taken out farther down by simple diversion works, thus saving considerable length of diversion or main canal. These conditions are found on a number of projects, such as the Tieton project in Washington, the Twin Falls Salmon River project, the Twin Falls South Side project, and the Boise project, in Idaho.

Smaller natural channels coming across the main canal can also be used in the place of laterals. They are supplied at their intersection with the main canal through headgate structures, and the flow is diverted from them to supply the distributaries by check-gate structures across the channel and distributary headgates at the head of the distributaries. This practice

has been used rather extensively on the distribution system of the Twin Falls Salmon River project and the Twin Falls South Side project, where the conditions are unusually favorable; these are: a comparatively steep slope, facilitating easy diversion from the channels, well-defined channels whose beds are composed largely of gravel and lava rock, which resist the high velocities. The advantages of their use is that under such favorable conditions, they will save considerable cost, they are safe channels, and usually give a small conveyance loss with, in some cases, a gain due to collected run-off of return waters from irrigation.

In general the practice is not desirable, especially if these channels are the natural outlet for run-off and underground drainage water and if the system is operated during the period when these channels may have to carry the natural run-off from rain or the melting of snows. These conditions will favor water-logging and make operation more difficult, and the channels will in many cases be required for the main arteries of a drainage system.

Waste and Drainage Channels.—On most projects the surface of the irrigable area is not smooth, without depressions or irregularities, although it may approach it in the lands of the plains or in the lower lands of large valleys. The irrigable land will usually form more or less well-defined depressions or distinct natural channels, in which drainage water will collect and if not obstructed will be carried to the main water course or stream. The drainage water may be produced by irrigation or by rainfall or by both, and may be either surface run-off or underground water. The surface run-off from irrigation may be the waste obtained at the ends of fields or the waste or surplus reaching the lower end of laterals and distributaries.

Surface waste by run-off at the ends of fields will vary with the method of irrigation and the skill and attention of the irrigator. With furrow irrigation and flooding in checks the waste can be practically eliminated. With the free flooding from field ditches, which is the prevailing method of irrigation in the Rocky Mountain States, the waste cannot be entirely prevented. Measurements on a number of projects in Montana give an average field waste of about 10 per cent. of the water applied. This waste is usually collected by small shallow drainage ditches at the lower end of the fields, and passes into other field ditches

or distributaries, or is conveyed to the natural water courses through larger drainage ditches which supplement the natural water courses.

The amount of waste or unused water at the end of laterals and distributaries will depend largely on the method of operation and the care used in the regulation of the flow in the different parts of the system. The distributaries can usually be operated and regulated so that all the water apportioned to them will be delivered and used, but on the laterals there may be times when an excess supply, resulting from variations in flow or from unused water, will reach their lower ends. Sometimes this water can be disposed of on worthless low land, but this practice is seldom desirable; usually all laterals and important distributaries should therefore be extended by what are called tail channels to discharge into water courses.

Underground water, due to irrigation, is produced by the seepage loss from the canal system and from the loss by deep percolation of the water applied to the land. The practice of irrigation usually causes a general rise of the water-table which on practically every project results in time in the damaging of considerable land by waterlogging and by the accumulation of alkali salts at the surface. The extent of this damage is largely dependent on the extent and drainage efficiency of the natural water courses.

Except on a few smaller systems consisting of concrete lined canals, flumes or pipe lines, where water is used very economically and with care, the natural drainage is not sufficient and on many projects complete drainage systems must ultimately be constructed. The location and design of a drainage system must be based on a careful study of the causes of waterlogging, and require an investigation of the soil, especially of the subsoil formations. The location cannot be based entirely on surface conditions; this is well illustrated by the occurrence on many projects of waterlogged areas where the surface conditions are apparently favorable for natural drainage. For these reasons the planning of a drainage system must usually be postponed until the needs for it are indicated by the appearance of affected areas. The principles of the design of drainage systems for irrigated lands are not presented, in order not to exceed the scope of this book. It will suffice to emphasize the importance of drainage by stating that various estimates of the land rendered worthless or damaged

by the rise of the water-table, with the consequent accumulation of alkali and waterlogging, place the total area on the projects of the arid region in the United States at 15 to 20 per cent. of the area irrigated.

Details of Location of Distribution System.—The location is preferably first made on maps of the topographic survey of the irrigable area and then laid out in the field, with minor modifications where necessary to adjust the lines more closely to the topography. The only variation from this procedure may be for rather unusual topographic conditions, such as regular slopes, pronounced well-defined ridges and depressions, when the paper location may be omitted, but as a rule a complete study based on a careful topographic survey will result in more economical and better location. The greatest value of a careful topographic survey may be not for the location of the main canal and laterals, but for the location of the distributaries, the elimination of high land areas, included within the boundaries of the distribution system but too high to be served, and for advising the settler in the preparation of his land for irrigation. In Vol. II, Chapter II, the methods of survey are considered.

In the location of the system on the topographic maps or paper location, one of the most important considerations is the necessity of adjusting the position of the laterals and especially the distributaries with respect to the ground surface, so that the water level in them is at a sufficient height above the ground surface to at least make deliveries to the high points or to the upper limit of the highland areas, and if possible to permit the installation of measuring boxes.

The distributaries should therefore be first considered. Usually at least 6 inches, plus the fall necessary to reach the high point on a flat grade, is desirable for this difference in elevation between the water level and the high point. To meet this requirement, the position of the bed of the distributary with respect to the ground surface and its grade is fixed by first plotting on the profile the required elevation of the water surface at the high points of deliveries and then connecting these by a line, which gives the grade line of the water surface in the distributary. Where this will bring the canal mostly in fill, as is often the case, it will usually be more economical to place the canal more nearly in balanced cut and fill with check gates at each point of delivery, building up the banks of the distributary for a short distance

upstream, to permit the backing up of the water. On most distributaries and practically always on those carrying a single irrigating head, a check gate is necessary at every delivery gate.

The laterals must be located and their position in cut or fill fixed to make the diversions through the distributary headgates with the required or desired difference in elevation between water levels. Where feasible, it is desirable to have the water level in the lateral, when operated at its minimum capacity (usually not less than ½ to ⅓ of its full capacity), at least 6 inches above the full supply water level at the head of the distributaries. This will largely eliminate the use of check gates, which are usually considered detrimental, in that the checking of the water increases seepage losses, encourages silt deposits, and may cause the overflowing of the banks resulting in some cases in breaks. As a rule, however, the elimination of check gates is only possible on laterals or portions of laterals which serve distributaries on steep grades, and in country with flat slopes, high knolls or high land areas it will usually be necessary to maintain the water level in the lateral by means of check gates up to its normal full supply level and to use a very flat grade for the distributaries. Under these conditions the minimum desirable difference in elevation between the full supply water level in the lateral and that at the head of the distributary may be taken as 6 inches.

The position of the canal section in cut or fill for a main canal or lateral is determined much in the same manner as for a distributary, by locating on the profiles the required water surface elevations at controlling high points, and from this the grade of the bed of the canal. Usually the main canal and the larger part of the laterals will be in balanced cut and fill section, and the distributaries will have a small excess of fill obtained by borrowing from each side and preferably from high points or high areas.

The following extracts taken from the instructions for the resurvey of the irrigable lands on the Lower Yellowstone project Montana, as stated by R. S. Stockton, the then Irrigation Manager, are of special interest as representing practice learned by experience on this project:

1. Determine if water will run from the given turnout to the high point of each farm unit or tract shown on the approved plots.

2. If some of the land is above water, make report as to whether same can be irrigated from another turnout or by raising banks of lateral a reasonable amount or by extending across an adjacent farm unit. The amount to be spent in such a case would depend on the acreage to be reclaimed. In deciding on the land above the water, assume that knolls not over 1½ acres in extent or ridges not over 200 feet wide will be levelled down by the owner.

3. If some of the land is above water or doubt exists as to watering any portion of the tract, run out the head ditches on a grade not flatter than ½ of a tenth to 100 feet. Assume the head ditches 1½ feet deep and carrying 1 foot of water, and for any considerable portion of the land, the water surface should be ½ foot above the ground surface, but does not need to be that much above the highest knoll or ridge when in 100 feet or so the water would be well above the ground in the field ditches. Where possible the head ditch grade is taken ½ foot below grade of lateral to take up loss of head in turnout (delivery gate), which might be, however, only $\frac{1}{10}$ or $\frac{2}{10}$ of a foot for a turnout box 12 inches by 18 inches delivering 3 second-feet of water. In cases where there is not enough fall and every bit of head must be saved, the grade of the ditch is taken out 1½ to 2 feet below the top of the bank of the distributary. This implies checking the distributary to within about ½ foot of the top of the bank for a fair head of water in the head ditch. Such a head ditch should be extra wide and deep, say 2 feet deep, so that in places where field lateral ditches take out from the bottom of the head ditch they will have a better flow of water.

Design of Canal Cross Sections for Distribution System.—The principles of design are included in the general discussion of canal cross sections, presented in Vol. II, Chapter VI. Smaller distributaries will usually have the forms and dimensions given for farm ditches in Vol. I, Chapter VII. It is especially important that distributaries and laterals have a large part of the water cross-sectional area above the ground surface; this is obtained with comparatively shallow wide ditches. An excess of fill will often be necessary, but its amount may be reduced by using minimum widths for the tops of the bank. The top width of bank is usually selected a little larger for a canal section all in fill than for one largely in cut, to give additional safety against breaks. The minimum commonly used for dis-

tributaries and small laterals is 3 to 4 feet. The freeboard for normal full supply flow in distributaries is usually 1 foot; with a minimum freeboard, when the flow is checked, of not less than 6 inches. In laterals the minimum freeboard when checked should be not less than $\frac{3}{4}$ of the normal freeboard. The usual side slopes are $1\frac{1}{2}$ to 1 and 2 to 1; the flatter side slopes will produce a smaller change in the depth of water for a variation in the flow.

Classification of the Irrigable Area and its Subdivision in Farm Units.—In order to command all the desirable irrigable area, it may be necessary to include within the boundaries of the distribution system a considerable area of non-irrigable land, or land which must be excluded from the irrigable area. There may be pieces of land or top of knolls which are too high to be reached, or areas of unfit or inferior land because either too rocky, too rough, too porous or charged with excessive amounts of alkali salts. In the area which is irrigable there will be certain parts occupied by fences, ditches, drainage channels, farm and county roads, railroads, rights-of-way, buildings, farm yards, schoolhouses, town sites, etc., and where the climatic conditions are favorable for dry farming, such as in certain sections approaching semi-arid conditions, there may be considerable areas in crops for which irrigation is not necessary or for which irrigation may be only desirable but not necessary except during dry years. There will also be on most systems, even when fully developed, areas which will remain idle for one or more seasons, and on nearly all systems the general rise in the water-table produced by irrigation results in the waterlogging of and the accumulation of alkali salts to an excessive amount on a small acreage, at first, but if not stopped by more economic use of water or by drainage, extending over a large part of the irrigated area.

Net Area to be Irrigated.—The system is usually designed for the net area to be ultimately irrigated. The land, which must be excluded on account of its elevation being too high, is best determined from the topographic survey of the area within the boundaries of the district and must be considered in connection with the location and elevation of the water level in the lateral or distributary. A rule used for a system in California is to consider as non-irrigable all land which could not be reached by a distributary on a minimum grade of 4 feet in 10,000, with

an extra fall of 6 inches. The land which must be rejected, because unsuitable or undesirable, must be determined from a physical and chemical survey of the soil. The ratio of the net area irrigated to the gross irrigable area included in the system will depend on a number of factors.

Mr. Don H. Bark, of the U. S. Department of Agriculture, found, from a survey of about 16,000 acres of well settled irrigated land in Idaho, that the net area of irrigated and cultivated land was 91 per cent. of the gross area. The surveyed area apparently did not include town sites, land made unfit by waterlogging or by the use of alkali, or land remaining idle or dry farmed. The result gives a per cent. of waste or non-irrigated land smaller than is generally assumed. A common estimate of the net irrigated land, obtained after full development, is 80 per cent. of the gross irrigable area, and when the climatic conditions are favorable for dry farming such as on certain systems in the Sacramento Valley of California where deciduous trees, cereals, beets will yield profitable crops in normal years without irrigation, the per cent. of net irrigated land may be considerably smaller.

Size of Farm Units and Plan of Subdivision.—The size of the farm units will affect the number of distributaries required to make the deliveries. On most projects and especially those of the Reclamation Service the average farm unit is from 10 to 20 acres for land planted to orchards and from 40 to 80 acres for land planted to alfalfa or diversified field crops. These have been found to be the most desirable size for new settlers with limited means.

The irrigable area is usually subdivided into square or rectangular farm units to conform with the section and ¼ section lines of the U. S. land surveys. This method facilitates the subdivision surveys and is well adapted to fairly uniform slopes, but when used where the topography is irregular so that many of the farm units are divided by ridges, depressions, or drainage channels, it will require more complicated and expensive farm distribution systems than for farm units adjusted to the topography, with upper and lower boundaries located along the ridges and drainage lines. Except on a few smaller systems of high priced land, the complications and difficulties of subdivision into farm units of irregular size adjusted to the topography have been regarded as too great to adopt this system of subdivision.

METHODS OF OPERATIONS OF DISTRIBUTION SYSTEM AS AFFECTING THE REQUIRED CARRYING CAPACITIES: CONTINUOUS FLOW—ROTATION FLOW—DELIVERY ON DEMAND

Principles of Methods of Operation.—When the entire distribution system is operated on the *continuous flow basis,* water is run in the main canal, laterals and distributaries all of the time during the irrigation season, and the flow during a period of deficiency is prorated to the different branches according to the area which they supply.

Operation by rotation may include:

First.—Rotation between the water users or groups of water users on each distributary, with continuous delivery at the head of the distributary; in which case the entire flow of the distributary is used in turn by the water user or groups of two or more water users for a period of time proportionate to their acreage.

Second.—Rotation between distributaries served by the same lateral combined usually with rotation between water users on each distributary. Usually the water carried by the lateral will be divided between groups of distributaries by operating the lateral in sections with the entire flow going to each section in turn. For instance, if a lateral serves 4,800 acres, with a capacity of 80 second-feet, it may be divided into four sections, each of which supplies a number of distributaries commanding a total of about $\frac{1}{4}$ of the total area or about 1,200 acres, each receiving the entire flow for $\frac{1}{4}$ of the time. This period of time may be 1 week out of every 4 weeks, or 5 days out of every 20 days or other suitable time periods, depending on the frequency of rotations. In each section the flow may be divided between all of the distributaries, supplied by that section, proportionately to the area commanded by each distributary, or may be divided and rotated between groups of these distributaries; for instance, if the desirable irrigating head is 16 second-feet, the flow may be divided and rotated between groups of five distributaries, each distributary receiving one irrigating head for a period of time proportionate to the area it serves, and the irrigating head in each distributary is assigned to each of its water users in turn for a fixed time per acre irrigated.

Third.—Rotation between laterals, by operating the main canal either continuously for its entire length, or by dividing it into sections, and with or without rotation between distributaries.

The method on the Modesto district has been to practise rotation between laterals, with rotation between distributaries, during the period when the available stream flow decreases below about ½ the capacity of the system.

Delivery on demand at the time and in such quantities as requested by the water user cannot be obtained in practice, but may be approached by the enforcement of certain restrictions which will give sufficient time to distribute the period of maximum demand over a longer interval. This may be obtained by requiring several days' notice from the water user and by limiting the amount of water he is entitled to receive. Without such restrictions, the required large capacity of the system as a whole would in most cases make the cost of the system prohibitive. The period of maximum demand will be shortest and most intense when a large part of the area is planted to the same crop and for shallow rooted crops, especially cereals which require irrigation within a short period at certain stages of their growth. Delivery on demand can be more nearly approached for diversified crops, with an ample water supply, a comparatively short diversion canal and a small system. It has been used successfully on a number of projects in Colorado, where a large part of the water supply was from storage reservoirs and a fixed quantity for the entire season was apportioned to each water user, with the privilege of using it at such times as he desires. It is frequently used during the early period of development of irrigation systems, when a comparatively small part of the irrigable land included in the project is irrigated.

Methods of Operation Used in Practice.—*Operation by continuous flow,* modified on a few systems by delivery on demand during the period of ample supply, is the prevalent method on the systems in Montana, Wyoming, Washington and Idaho. In systems more recently constructed in these states, especially several in southern Idaho and some of those of the Reclamation Service, and on a number of older projects, operation by rotation between water users has been introduced to a limited extent.

Operation by rotation is probably the prevalent method in Utah, California, New Mexico and Arizona. On the Bear River Canal system in Utah rotation is practised on the distributaries; the water user receives the water once a week; a minimum irrigating head of 2.1 second-feet is delivered for 1 hour for each acre, but he may use and often does use double the head for half the

time; he confines the flow of each week to about ¼ of his acreage, thus requiring four turns to irrigate his entire holding; this gives him an average of 8.4 inches depth of irrigation, equivalent to 9 inches per month. On this system in 1908 about 45 per cent. of the irrigated land was in cereals, 22 per cent. in alfalfa, and 15 per cent. in sugar beets. The diversification of crops on this project, with a considerable acreage in at least one deep-rooted crop such as alfalfa, is especially favorable to the rotation practice adopted.

On the Huntley project in Montana, with a crop distribution similar to that given above but with a smaller acreage in cereals and larger acreage in beets, rotation at 1-week intervals was used at first and later changed to 4-day intervals. The rotation practices on this project and on the Bear River Canal system are typical and applicable to systems in Montana, Utah, Wyoming and elsewhere, which serve land planted to diversified crops, with the larger acreage in annual crops such as cereals and sugar beets, with a smaller area of alfalfa, and irrigated by free flooding from field ditches, with the usual irrigating head of 2 to 4 second-feet.

On the Tempe Canal system in Arizona, rotation is practised on the distributaries between water users. The irrigating head is 10 to 15 second-feet and is delivered to the water user, every 6 to 10 days, when the river is normal, for a period of time depending upon the number of shares he owns. On the Salt River project in Arizona delivery is made by rotation once in 8 days, continuing from 6 to 12 hours for 40 acres, with an irrigating head of 10 second-feet. Of the area irrigated in 1911, 55 per cent. was in alfalfa and 28 per cent. in cereals. The prevalent method of irrigation is by flooding with border checks. The rotation practices for these projects are typical for the southwest for systems serving land planted largely to alfalfa with a considerable area of cereals, irrigated by the border check flooding method.

On the Modesto irrigation district system in California rotation is practised as previously stated. The irrigating head is about 20 second-feet and is delivered to the water user once in 4 weeks for a period of 20 minutes to the acre. About 75 per cent. of the land is in alfalfa. Border checks and rectangular checks are used almost exclusively. The rotation practice on this system is representative of the practice on a number of projects in the San Joaquin and Sacramento Valleys, where the area in

alfalfa is about 75 per cent. of the total area irrigated and where shallow rooted crops are of minor or no consideration. The interval between irrigations of 4 weeks may be longer than is desirable, especially for porous sandy soils.

On the irrigation systems of southern California, supplying citrus orchards or walnut orchards, with in some cases a smaller acreage in alfalfa, operation by rotation between water users and usually between pipe distributaries is necessary on account of the small orchard units, usually not over 10 acres, and the high duty, which on a continuous flow basis would give a stream too small to irrigate with. A common allotment is equivalent to 1 miner's inch continuous flow to from 5 to 7 acres; the general practice is to accumulate the right to the flow for a period ranging from 30 to 60 days, usually about 30 days, and use a head of from 25 to 100 miner's inches (½ to 2 second-feet), usually 40 to 60 miner's inches, for a length of time depending on the acreage, generally of 24 hours for each 10 acres.

Selection Between Operation by Continuous Flow and Rotation.—The concensus of opinion of irrigation engineers and superintendents is that operation by rotation is in general preferable, and its use is gradually being extended to replace continuous flow operation. There is no doubt that it can be applied advantageously to a large number if not the majority of irrigation systems on which operation by continuous flow is now practised; but there are certain conditions which will require continuous flow operation.

The conditions most favorable to continuous flow are an ample water supply, comparatively large farm units to give allotments of water sufficient for economical irrigation heads, with which rotation can be practised on each farm unit, irrigation methods which require a comparatively small irrigating head, soil texture or topographic features which require a small head. Some of these conditions are obtained on many of the systems in Washington, Oregon, Idaho, Montana, Wyoming, and British Columbia, where the land served is either steep and rolling, composed of light soil easily eroded, such as some of the orchard foothill lands in Washington, which require irrigation by furrows with small heads; or where the land is divided into comparatively large holdings of 80 to 160 acres or more, in which case each farm unit will have an allotment of at least 1 to 2 second-feet continuous flow. On some of the older systems with ample water

supply, often protected by excessive court decrees, comparatively large irrigation heads may be obtainable even for smaller farm units, but this usually results in the head being used only part of the time and wasted the rest of the time. In greater detail the conditions most favorable to and the special advantages of continuous flow are:

1. When a canal system includes considerable wooden fluming, in which case a continuous flow is desirable to keep the flumes water-tight and in good condition and to prolong their life.

2. On heavy clay soils, subject to cracking or drying, in which cases a continuous flow in the ditches may produce a smaller seepage loss and fewer breaks than a rotation flow.

3. Continuous flow requires smaller distributaries and structures and in some cases smaller laterals. This advantage is, however, very small, for usually the distributaries must be made of a certain minimum capacity which would permit rotation.

4. Continuous flow is more convenient when domestic and stock water is obtained from the irrigation system. This is the practice on a number of projects, where ground water is not easily obtainable; but usually only during the early period of settlement. For these conditions rotation delivery requires the use of storage tanks. The disadvantages of continuous flow operation are indicated by the advantages of operation by rotation.

The conditions which make operation by rotation most desirable are small farm units, irrigation methods which require comparatively large irrigating heads, and periods of deficient flow. Other controlling factors in the selection of the method of operation are the following advantages:

1. The irrigating head which it gives is usually the size of stream best adapted to the prevailing method of irrigation, and is favorable to equal distribution of water on the land, with a minimum loss by deep percolation. It also decreases the seepage loss in the conveyance of water over the farm, because of the smaller proportionate loss in conveying a large stream, and makes it possible for the water user to irrigate his farm in a comparatively short time.

2. The deliveries are usually made according to a fixed schedule which enables the water user to plan his other work and carry on his farming operation in a businesslike way without continuous

interruptions to attend to irrigation. This makes him appreciate the full value of the water and the large head will usually make it necessary for him to devote his best efforts to its proper use with a minimum of surface waste, unless the head be too large for him to handle properly.

The decrease in loss by seepage, deep percolation, and surface waste will reduce the time of irrigation by a greater ratio than the increase in head. For instance, with an irrigation head of 2 second-feet in free flooding from field ditches, the time of irrigation will be less than ½ of the time required with a head of 1 second-foot.

3. Rotation between distributaries and between laterals or sections of laterals will usually decrease the conveyance losses of the distribution system and will give periods when the ditches are dry, during which repairs and maintenance work can best be done. Moss and aquatic plants will be better controlled, if not killed during the dry period.

4. The cost of operation will be materially decreased, especially by rotation between groups of distributaries, which concentrates the work of the ditch tenders.

5. A fewer number of structures for the measurement of water are necessary, because many of the distributaries will each carry a single irrigation head, which can be measured by a single measuring structure at the upper end of each distributary.

On systems where the change is made from continuous flow operation to rotation operation, dissatisfaction may result at first, but if the system is made rather elastic during the early stages of its introduction, and if the irrigating head and interval between irrigations be adjusted to the irrigation method and crops, the results will be a more economical use of water and a feeling of equal treatment of the water users.

CARRYING CAPACITY OF DISTRIBUTION SYSTEM

General Considerations.—The capacity of the irrigation system, considered as a whole, must be based not only on a careful consideration of all available data on the duty of water, especially seasonal duty, but on estimates of the maximum use of water. The variations in use during the irrigation season, as far as monthly use is concerned, are represented by the monthly seasonal duty, but estimates of capacity based on data of the maximum monthly use may in some cases be considerably less

than the desired maximum capacity during a portion of a month. The maximum use for which the system must be designed is limited by the character of the water supply and economic considerations.

When the supply is deficient and obtained from a stream which is subject to sudden variations in flow, which are not regulated by storage, then there will often be a large flow for a few days which may be used to advantage for a heavy irrigation, especially if a period of deficient flow is to follow. This condition will justify making the canal system comparatively large to benefit from the flood flow, obtainable only for a few days. The extent to which the additional cost of constructing a system of large capacity is warranted is based on the dependability of these freshets and on the value of the surplus water thus obtained. In general, the surplus flow from freshets or short period floods can be utilized to a smaller extent on large systems than on small ones, because of the operation difficulties resulting from sudden variations in flow. This type of water supply is obtained from streams whose flow is largely dependent on rainfall run-off, or from streams whose watershed is at a comparatively low elevation, on which the accumulated winter snow is subject to rapid melting. These conditions are obtained on a number of streams in the southwest states and many smaller streams in practically all states.

Systems depending on such irregular and deficient stream flow, not regulated by storage, will often show a relatively high average duty, but the maximum rate of use during the short period of large stream flow may be 2 or 3 times the average maximum monthly rate of use. For instance, measurements taken by the Irrigation Investigations of the U. S. Department of Agriculture, for the season of 1899–1900, on a number of systems in the Salt River Valley, Arizona, prior to regulation by storage, gave an average daily use for the month of maximum use of about 501 second-feet for 90,000 acres, or about 1 second-foot to 180 acres, while the maximum use during the month was at the rate of 1 second-foot to 66 acres. A similar condition of sudden fluctuation in use is obtained when the growing season is short and a large part of the irrigated land is planted to shallow rooted crops which require the heaviest irrigation at about the same period of growth and within a short time. This is especially noticeable on some of the systems in the Rocky

Mountain States where a large part of the land is planted to cereals, and is less noticeable where alfalfa with orchards or diversified crops are grown.

When the supply is ample throughout the season, or when there are no sudden fluctuations in the stream flow and the land is planted to diversified or deep rooted crops, then there will be no sudden variations in the use of water, and especially for larger systems the maximum use will be very nearly the same as the average daily use for the month of maximum use.

The capacities to be considered in the design of the system are:

First.—The capacity of the diversion canal at the point of diversion.

Second.—The capacity of the main canal at the head of the distribution system.

Third.—The capacities of the laterals.

Fourth.—The capacities of the distributaries

Capacity of Diversion Canal.—The capacity of the diversion canal is usually made the same from the upper end at the point of diversion to the lower end at the head of the distribution system, except in some cases when for a short distance the upper end is given a surplus capacity down to the first escape or sand sluice; this may be necessary for closer regulation of the flow or to scour out material deposited in this section of the canal. Where the diversion canal is located along steep side hills, with rough rocky irregular topography, requiring expensive construction with numerous flumes or siphons, greater economy may result by using for the first years of development, temporary wood flumes built only for part of the ultimate capacity; but for rock excavation and permanent concrete construction it will generally be more economical to construct for the ultimate capacity, obtained from generous estimates in order that no expensive enlargements be necessary.

The capacity of the diversion canal will be based on estimates of the maximum use as determined from the study of the gross monthly seasonal duty on similar projects and the factors affecting the maximum use as stated above. Estimates may also be based on a similar study of net monthly seasonal duty, to which must be added an allowance for the conveyance losses. The conveyance loss will gradually decrease the flow in the canal system from the head of the diversion canal down to the points of delivery; to correspond with this, the capacity of the diversion

canal could be made decreasing, but as the estimates of conveyance losses are necessarily uncertain and as they will decrease either by the natural process of silting or by improvements made to stop them, the usual and desirable practice is to make the capacity of the diversion canal uniform from its upper to its lower end, except for unusually long canals.

Capacity of Main Canal.—The capacity of the main canal at the head of the distribution system will be about the same as that of the diversion canal. The flow in the main canal and in the main laterals diverting from it will usually be continuous, so that the capacity of the main canal may be decreased below the head of each main lateral by the normal supply diverted by that lateral. It is desirable to make no reduction in the carrying capacity for the expected seepage loss. The capacity thus obtained for each of the different sections of the main canal will then permit carrying a surplus equal to the seepage loss. This small surplus capacity is desirable to allow for the variations in flow produced by the regulation of the main lateral headgates.

Capacity of Main Laterals.—The main laterals are in most cases operated continuously through their entire length, but there are several projects, notably some in the San Joaquin and Sacramento Valleys where rotation of flow is practised on the main laterals, by dividing each lateral into sections which will include the heads of groups of sub-laterals or distributaries whose combined capacity must equal the capacity of the main lateral.

When a main lateral is operated continuously down to its lower end, its capacity may be diminished below the heads of sub-laterals and distributaries, but it must be designed for a comparatively lower duty than that of the main canal in order to obtain sufficient elasticity for variations in flow produced by the closure or regulation of headgates.

When rotation is practised on the main lateral, the capacity of the main lateral must remain the same down to at least the beginning of the last rotation section, from which point it may be diminished down to required size of waste channel at the lower end. On the Modesto irrigation system this method of operation is used on all the main laterals, and rotation between laterals is practised when the stream flow decreases to less than about one-half the full supply capacity of the main canal; this, however, does not require any larger capacity than that required during the full supply period.

The capacities of the sub-laterals are obtained on the same basis as the main laterals when operated continuously, and on the same basis as distributaries when operated by rotation.

Capacity of Distributaries.—The distributaries are usually designed for operation by rotation, in which case the carrying capacity of each distributary will depend not only on the area and number of farm units served, but also on the period of time that the water is in the lateral. For instance, if a distributary receives water for ½ of the time, its capacity must be twice that obtained on a continuous flow basis.

When the distributary is to be operated by continuous flow, its capacity is determined from a duty at least equal to and preferably lower than that of the laterals in order to permit the use of a surplus for short periods.

The minimum capacity of any distributary must be not less than a satisfactory irrigation head, and when larger must be preferably a multiple of the irrigating head. The size of the irrigating head will depend on the method of irrigation. For furrow irrigation it will range from a fraction of a cubic foot per second to usually not over 1½ second-feet; a satisfactory head is about 1 cubic foot per second. For the field flooding method it will range from about 1 to 4 second-feet; 2 second-feet is a desirable head. For check flooding a head less than 4 second-feet is seldom used, although with small checks 2 second-feet and even 1 second-foot may be used; usually the head will range from 10 to 20 second-feet; a satisfactory head is 14 to 20 second-feet.

Suggested Values of Carrying Capacities Based on Practice.—The above considerations show that the capacities of the different parts of the system are all related so that the combined capacity of the distributaries diverting from a lateral must be at least equal to and usually greater than the capacity of the lateral; the same relation must exist between the combined capacity of the laterals diverting from a main canal and the capacity of the main canal.

The carrying capacity of every part of the system must be based on the area supplied and on the considerations presented above. The estimates should be made after a complete study of the seasonal duty, conveyance losses and other factors on projects as nearly similar as possible. From a study of the maximum use of water on a number of irrigation systems, the

following general values are suggested as representing average practice. These values must be used with caution and only in connection with the comprehensive study outlined above.

Suggested Values for Maximum Carrying Capacity of Diversion Canal or Main Canal, Expressed in Number of Acres of Irrigated Land per Cubic Foot per Second

1 second-foot to 40–60 acres, where supply is dependent largely on short periods of flood flows, without storage, such as on some projects in New Mexico and Arizona, or where a large part of the area irrigated is cereals or other shallow rooted crops, creating a short period of maximum demand, such as on some smaller projects in the Rocky Mountain States.

1 second-foot to 60–70 acres, used on a number of large systems in the San Joaquin and Sacramento Valleys, California, where the stream flow is usually ample for the first half of the irrigation season, and for similar conditions in other states.

1 second-foot to 70–80 acres, used on most systems, throughout the arid region, where water supply is fairly regular; for area irrigated largely in diversified shallow rooted crops with some deep rooted crops, such as in the Rocky Mountain States of Wyoming, Montana, Utah and Colorado.

1 second-foot to 80–100 acres, used on many systems, throughout the arid region, where water supply is regular, either naturally or by storage; for area irrigated largely in orchards and alfalfa, and a growing season comparatively long, such as on a number of large systems in southern Idaho, eastern Washington, Oregon, Arizona and California.

1 second-foot to 120–160 acres, used on the majority of the larger systems of southern California, consisting of concrete lined canals and pipe-line distributaries; for area irrigated in citrus fruits, deciduous fruits and alfalfa, and on available water supply which is relatively small and must be carefully used.

In general the smaller acreage in the suggested values should preferably be used when the conveyance loss is expected to be comparatively large and for relatively small systems, operated either on the demand basis or on the continuous flow basis.

The laterals are usually designed for relatively larger maximum capacities to give a certain amount of elasticity in operation. The excess capacity to provide this elasticity in a lateral should be a certain percentage of the flow of water actually delivered at the head of the lateral—usually about 40 per cent. of this flow. But the rate of flow delivered is less than the rate of flow diverted at the head of the diversion canal by the conveyance loss in the diversion canal. Therefore the maximum capacities for the laterals may be based on the suggested rates of maximum capacities given above for diversion canals, decreasing the areas of those rates about 10 per cent., where a comparatively large conveyance loss of 20 to 30 per cent. is expected and about 25 per cent. where a small conveyance loss is expected.

The distributaries are usually designed to permit operation by rotation, in which case their capacities will be based on a rate in which the acreage per second-foot will be about ½ that of the main canal; the capacity must never be less than at least one irrigation head and preferably two. Mr. H. N. Savage, formerly supervising engineer for the Northern Division of the Reclamation Service, which includes Montana, parts of Wyoming and North Dakota, states that in many projects of this division the main canals have been designed for a capacity of 1 second-foot to 80 acres, the laterals for 1 second-foot to 60 acres, and the distributaries for 1 second-foot to 40 acres, but not less than 4 second-feet.

CHAPTER IX

CHECK GATES

Object of and Types.—A check gate is a structure placed across a canal to control the flow and depth of water in the upstream section of the canal. It is generally used to raise the water level in the canal in order to divert part or the entire flow through one or more lateral headgates on the upstream side, or to stop the flow of water down the canal and divert it through a wasteway or sluiceway.

A check gate may be built as a separate structure, but is often combined or built with another structure. It is frequently built as a combination structure with a lateral headgate on one side or on each side upstream of it, and is then commonly called a division box, at least when the structure is small such as when used on the laterals of the distribution system. Drops, especially those on the distribution laterals, often occupy a position favorable for a check gate, and the structure is then built as a combined check gate and drop by forming the check gate at the inlet to the drop. A sluiceway or wasteway will often be built to advantage as one structure with a check gate just below it.

The use of check gates is primarily to control the depth of water in the canals of the distribution system. These canals are usually built partly in cut and partly in fill, approximating a balanced cut and fill section. This section with the canal full will hold the water level above the natural ground surface; the proportion of the volume of water thus carried above the ground surface depends on the form of the cross section and has been shown to be greater for broad shallow canals than for narrow deep canals under similar conditions (Vol. II, page 118). But there are always times when canals are operated at partial flow; the water level may then drop below the ground surface, in which case check gates are necessary to raise the water level to deliver water through the headgates above. When the land surface slopes down away from the canal at an inclination

sufficient to bring the ground surface lower than the low water supply level in a comparatively short distance, the necessity for check gates is not so great; but unless the waste strip of land adjacent to the canal is not to be considered, it will usually be desirable to have check gates; however, they will be placed at farther intervals than where there is little slope away from the canal.

Check gates may be divided into classes of two general types: the overpour and the undershot. The overpour type regulates the depth of water in the canal by horizontal flashboards, over which pours the water not diverted above. The undershot type regulates the flow with gates which allow the water to pass under them. The special advantages of each are the following:

First.—The overpour check gate is better adapted to the measurement of the water.

Second.—The undershot check gate is usually easier to operate, as it involves less labor than the removal of flashboards.

Third.—The increase or decrease in the depth of water in the canal above the check gate resulting from the variations in the flow of the canal due to an increase or decrease in the volume of water delivered through the lateral gates above the check gate will be smaller with the overpour type than with the undershot type. This is illustrated by the following example:

Consider a canal of the following dimensions and carrying capacity: Bottom width, 10 feet; depth of water, 5 feet; side slopes, $1\frac{1}{2}$ to 1; grade, 5 feet in 10,000; $n = 0.025$; full supply carrying capacity, 300 cubic feet per second. Assume that the canal is being operated at $\frac{1}{2}$ full capacity or 150 cubic feet per second, of which 50 cubic feet per second are being diverted through one or more lateral headgates upstream from the check gate; the remaining 100 cubic feet per second passing through the check gate into the section of canal on the downstream side of the check gate, and that the water is maintained to the full supply depth of 5 feet at the check gate. If flashboards are used for the regulation, and if the net length of flashboard between side walls is 15 feet, the depth of water pouring over the crest for a flow of 100 cubic feet per second is by the weir formula 1.59 feet. It may now happen that the lateral headgates above are shut without changing the flashboards in the check gate; the result will be a flow of 150 cubic feet per second at the check gate, which will increase the depth of overpour to 2.09 feet producing a

raise of 0.50 feet in the water level above the full supply depth. If the check gate is regulated with undershot gates, of a net width between side walls of 15 feet, the gate openings will be submerged. For a flow of 100 cubic feet per second through the gates, the depth of water in the canal below will be 3.1 feet and with the water level on the upstream side of 5 feet the required height of opening for a coefficient of discharge of 0.7 is 10.5 inches. Assuming as with the flashboard gates that the lateral headgates above are shut and that the increased flow of 150 cubic feet per second must pass through the unchanged gate opening, then the depth of water in the canal below the check gate increases to 3.9 feet and the water level on the upstream side increases to 4.25 feet above the downstream water level, producing a depth of water on the upstream side of the gate equal to 3.9 + 4.25 or 8.15 feet.

These results show that the excess flow produces an increase in water depth on the upstream side of the check gate of only 6 inches with flashboard regulation as compared with 3.15 feet with undershot gate. This comparison indicates the advantage of the overpour check gate where it is to be used for the purpose of maintaining a uniform water level in the canal above the check gate. On the other hand, where the checkgate is used to check the flow of water to divert part of it into a wasteway or sluiceway, allowing the required quantity to go through the gate down the canal, the undershot type will maintain a more constant flow in the canal, because the discharge through the gate is less affected by an increase or decrease in the depth of water above the gate opening.

Principles of Design of Check Gates.—The type of structure, whether overpour or undershot, will be determined largely from the above considerations. Usually it should be of the overpour type. On many projects the form of the gates and makeup of the structure will be very similar to that used for the headgates of the main canal, main laterals and sluiceways. For instance, on some of the older projects a common type of structure consists of wooden frames, which divide the total width of the structure into panels or openings regulated by horizontal flashboards. On other projects the gates will be vertical lift gates of the same type for all structures; on others radial or Taintor gates are commonly used. The features and principles of structural design are similar for all these structures and are included in the

standard principles of design previously presented in the discussion of headgates.

The hydraulic design involves a consideration of the hydrostatic pressures and the effects of the flowing water. The flow through or over the gates and the effect of the variations in the canal flow have been discussed above.

In the overpour type of gate, the flashboards are inserted either in vertical grooves or in grooves sloping downstream on a slope varying from about 3 feet horizontally to 5 feet vertically, to about 5 feet horizontally to 7 feet vertically. The advantages claimed for a sloping face are:

First.—The vertical downward component of the hydrostatic pressure increases the stability of the structure, which is specially necessary for light wooden structures.

Second.—The upper flashboards have a lesser tendency to float up when the water level on the downstream side rises up to nearly the water level on the upstream side.

Third.—The flashboards can be made more water-tight because silt or other transported material has a better chance to catch in the cracks between flashboards.

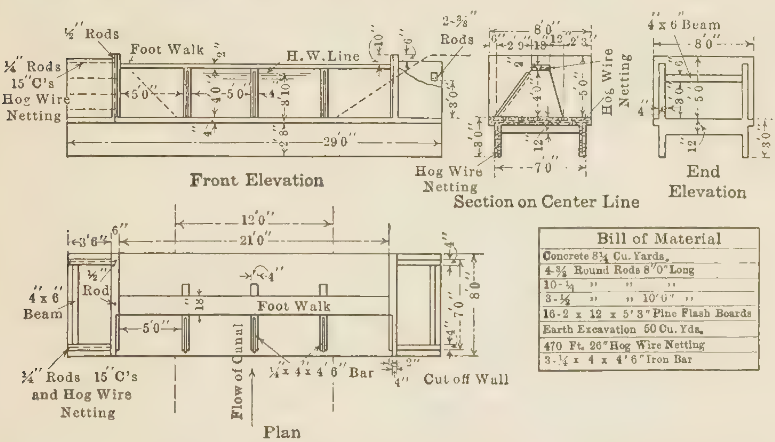
The floor on the downstream side must be made at least sufficiently long to receive the sheet of overpouring water at about its center; this will be obtained, as for a drop, on the assumption that the full depth of water in the canal is the maximum height of fall and that a surplus flow of 1 foot in depth over the crest is liable to occur; this will give a length of downstream floor, measured horizontally from the crest, equal to $3.0\sqrt{D}$ where D is the full depth of water.

With both types of check gates it is desirable not to make the net width of opening between the side walls of the structure too small, for the contraction has a tendency to produce cross currents or eddies at the outlet, increasing the erosive effect. Usually a width equal to about the average width of the canal is used. The floor should be placed at least 6 inches and preferably 1 foot lower than the canal bed, in order to form a shallow water cushion and so that it will be not higher than the canal bed on the downstream side in case there should be some erosion of the canal bed downstream from the structure.

When the material in which the check gate is built is porous, there is a greater tendency for the water to find its way around or under the structure, which may cause also an uplift pressure

on the underside of the floor of the structure. To make the structure safe against the underflow or flow around the sides, the path of creep or percolation must be made equal to 4 or 5 times the full water depth for clay loam soils, and 6 to 8 times for loamy soils and sandy soils. The depth of cut-off wall at the upstream end is usually made equal to not less than half the depth of water for average sandy loam soil, and not less than the entire depth of water for open sandy or gravelly soil. The uplift pressure must be considered with the other hydrostatic pressures, and the weight of the structure and the design should give a resultant which will fall within the middle third of the base.

Flashboard Check Gate on Pawnee Canal of Arkansas Valley Sugar Beet and Irrigated Land Co., Colorado (Fig. 116).—This



Bill of Material
Concrete 8¼ Cu. Yards.
4-⅜ Round Rods 8′0″ Long
10-¼ ″ ″ ″
3-½ ″ ″ 10′0″ ″
16-2 x 12 x 5′ 3″ Pine Flash Boards
Earth Excavation 50 Cu. Yds.
470 Ft. 26″ Hog Wire Netting
3-¼ x 4 x 4′ 6″ Iron Bar

Fig. 116.—Checkgate on Pawnee Canal. Arkansas Valley Sugar Beet & Irrigated Land Co.

structure is built on a canal whose capacity is 200 cubic feet per second and represents a good type of reinforced concrete check gate. The channel formed between side walls is divided by three concrete buttresses, 4 inches thick, into four bays each 5 feet wide. The net width of the structure is a little greater than the average width of the canal, which is a desirable feature. The flashboards are supported at their ends on sloping shelves formed in each of the side walls and on the upstream face of the buttresses, from which projects the edge of an iron bar, partly imbedded in the concrete, which divides the face

of the buttress in two parts to guide the placing of the flashboards. The flashboards form a sloping face, on which the hydrostatic pressure has a downward component, which is specially necessary in this case to increase the stability of the light structure, formed of thin reinforced concrete walls and floor. The floor is extended beyond the junction with the side walls, and with the side walls and wing walls at each end holds a wedge of earth, which increases the weight of the structure. The wing walls are connected across by a reinforced concrete tie beam, which increases their stability against earth pressure. The walls and floor are comparatively thin and reinforced with hog

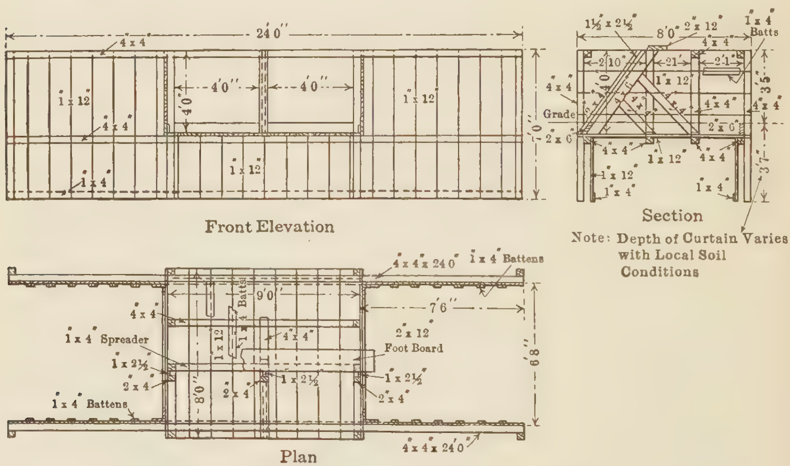


FIG. 117.—Timber flashboard checkgate. Sacramento Valley Irrigation Co., Calif.

wire netting with additional rods in the side and wing walls. The reinforcement in the floor gives it sufficient strength to act if necessary as a floor slab divided into spans by the buttress walls against an uplift hydrostatic pressure on the under side of the floor.

Timber Overpour Flashboard Check Gate of Sacramento Valley Irrigation Company, California (Fig. 117).—This structure illustrates a type very commonly used on small and large laterals. In this case the structure forms two openings each 4 feet wide, regulated with flashboards placed on a slope of about 3 feet horizontal to 4 vertical. The width of the structure between side walls is made about equal to the average width of the

canal. The floor is placed 6 inches below the canal grade, thus forming a water cushion of that depth. The top of the structure is built up to the top of the banks and the cut-off side wings extend at least to the center of the crown of the bank.

Flashboard Check Gate on Boise Project, Idaho (Fig. 118).—The channel between side walls is 43 feet 6 inches wide, divided into seven bays by six reinforced concrete buttresses, which are anchored by the reinforcement to the floor. This anchorage is partly necessary to hold the buttress against a slight cantilever action which will be obtained from the maximum hydro-

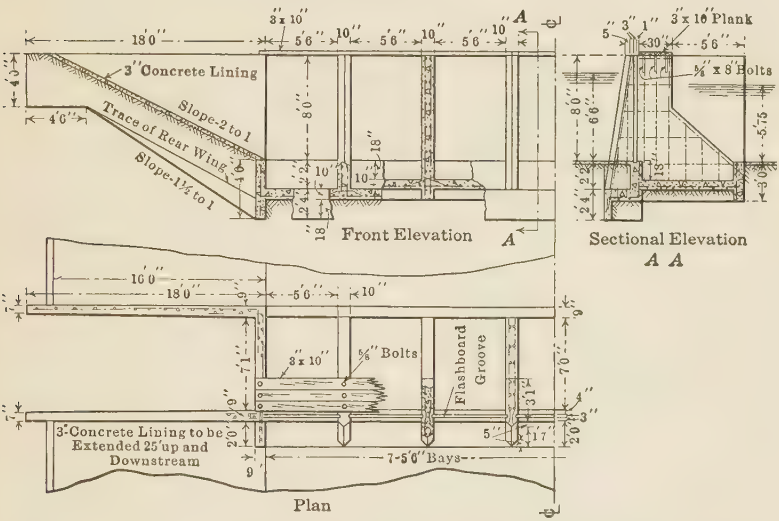


FIG. 118.—Concrete check gate. Boise Project, Idaho.

static pressure when the water level upstream is at full depth with no water on the downstream side.

The upstream portion of the floor is depressed below the bed of the canal and holds 2 feet 2 inches of earth which gives the structure added stability and greater safety against underflow. The downstream floor is placed 18 inches below grade to form a shallow water cushion, which also adds to the weight of the structure, as there will be enough leakage through the flashboards to keep it full under all conditions. The inlet and outlet to the structure have been formed by a concrete lining 3 inches extending upstream and downstream for a distance of about 25 feet. The total width of the channel between the side walls

of the structure is equal to the bottom width of the canal; it would be preferable to make it equal to about the average width of the canal to avoid the contraction in water area.

Cost of Flashboard Check Gate, Boise Project

Camp maintenance, labor and supplies	$45.00
Preparation, expenses of assembling plant	47.16
Excavation, backfilling and puddling, 300 cubic yards	259.65
Concrete:	
Sand and gravel, including open gravel pit, screening, hauling	189.32
Cement: 387 sacks at 64 cents	247.68
Hauling	20.15
Water for concrete	52.62
Forms: Lumber 2,112 B.M.	37.24
Labor	97.57
Mixing and placing concrete, 90.7 cubic yards	217.78
Reinforcement: Steel 2,823 feet of 5/8 inch, 1,786 feet of 3/8 inch	112.73
Labor of placing and hauling	36.09
Repairs to equipment and depreciation	12.81
Engineering, superintendent and accounts	108.34
Operating bridge: Labor	8.57
Supplies	6.24
	$1,499.13

The check-gate structure alone contains 51.4 cubic yards of reinforced concrete with 204 sacks of cement; the lining contains 39.3 cubic yards of concrete with 183 sacks of cement. To permit laying during freezing temperatures, 3⅛ pounds of salt were used for each cubic yard of concrete lining.

Cement was hauled 5½ miles, water ½ mile, gravel and sand 2½ miles, steel 12 miles.

Undershot Radial Check Gate of the Yolo Water and Power Co., California (Fig. 119).—The radial gate or Taintor gate, a simple form of which is illustrated in this structure, has been adopted on a number of the more recent irrigation projects. Because of the undershot flow, it is, however, better adapted for use on headgate or delivery gate structures than for use on check-gate structures. It has many advantages over the common form of straight lift rectangular gate placed in vertical grooves. It is more easy to operate, requires a comparatively small lifting force, and will permit the use of wide gate opening, which would be a special advantage in localities where canals must be operated at a time of the year when water carries ice. A cost comparison of this special design, with a structure divided by piers into smaller

openings regulated by vertical rectangular gate of the ordinary type, showed that for the same character of construction the radial gate type was less expensive.

The structure is used on a concrete lined canal of the Yolo Water & Power Co. The radial gate consists of a curved wooden face supported against the three curved ribs of the framework and braces, which transmit the pressure to the axle. The framework is built of structural angles and the wooden facing is made of horizontal boards, planed with radial edges. To press the facing against the curved ribs and also to obtain water-tightness, iron bands are placed opposite each rib

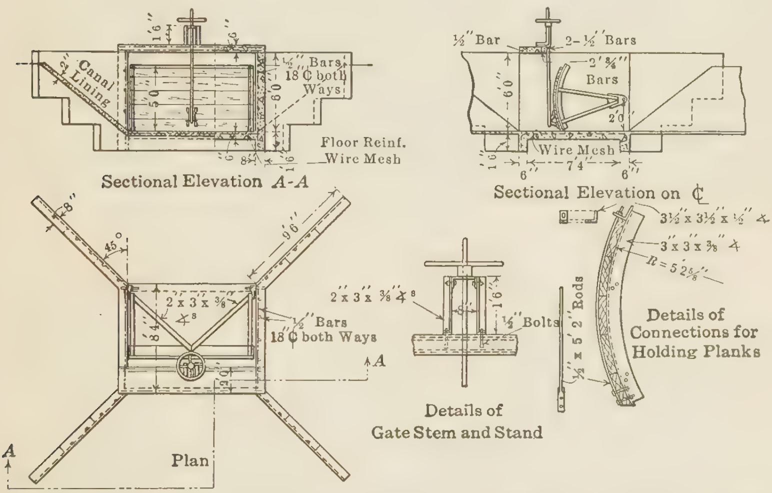


FIG. 119.—Undershot radial check gate. Yolo Water & Power Co., Calif.

on the upstream side of the facing; one end of each rod is flattened and is bolted through the bottom plank to the lower end of the rib; the other end is threaded and connected to the upper end of the rib by means of a short piece of angle with a slot which permits drawing the boards together by screwing the nut on the threaded end. To prevent leakage between the sides of the facing and the side wall, rubber belting is fastened to the edges of the boards and bent so as to press against the side walls.

The lifting device is pivoted just below the operating wheel to permit the necessary swing of the gate stem.

Combined Undershot Radial Check Gate and Drop on Lateral of University Farm, California (Fig. 120 and Plate XV, Fig. A).

This structure is built on a comparatively small lateral. The radial gate consists of a curved wooden face built on a timber framework, which bears against and is bolted to a special casting used for axle bearing. The axle passes through wooden blocks,

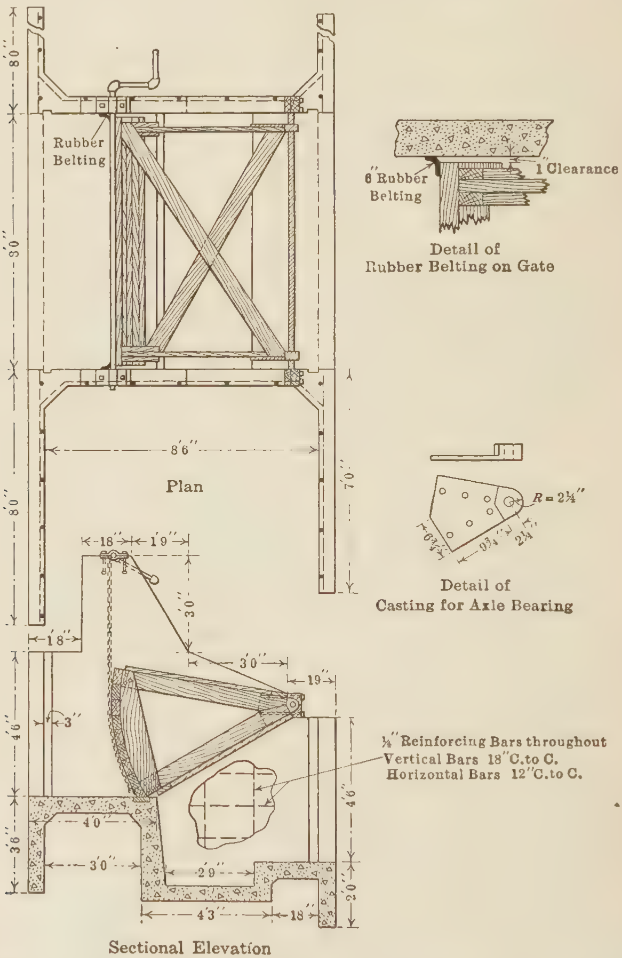


FIG. 120.—Combined headgate and drop, University Farm. Davis, Calif.

which are bolted to the side walls. The wooden blocks were used to adjust the position of the axle so that the gate would fit closely between the side walls. The lifting device is of the windlass type, formed of a 2-inch galvanized iron water pipe, around which the lifting chains are wound, and of an operating

Fig. A.—Combined undershot radial check gate and low drop. University Farm, Calif.

Fig. B.—Automatic undershot radial check gate. Turlock Irrigation District, Calif.

(*Facing page 312*)

lever arm connected to the pipe with standard elbows. The pipe revolves in a special bearing formed of plates bolted to the concrete standards. In the place of these standards each support could be formed of a section of pipe embedded at the lower end in the concrete side walls and connected at the upper end to a Tee fitting, of such size that the axle pipe would pass through it.

Automatic Undershot Radial Gate used on the Canals of the Turlock Irrigation System, California (Fig. 121 and Plate XV, Fig. B).—A number of these structures have been installed to maintain automatically an approximate constant depth of water in the canals. This was deemed desirable in order to maintain more uniform deliveries through the delivery or take-out gates and to prevent the trouble caused by the burrowing gophers; because when the canals are operated at partial flow with a corresponding small water depth, the gophers burrow in the bank above the water level and form channels, which may result in considerable loss by leakage and in breaks when the depth of water is increased to full depth.

The channel of the structure between the side walls is divided by buttress walls into three or more openings or bays. The main opening in the center is regulated by the automatic radial check gate, and the smaller openings on each side are regulated by flashboards, over which the water pours. The radial check gate is formed of a curved wooden face built of a double layer of 1 × 6-inch boards with lapped joints, nailed to the wooden ribs of the framework. The radial arms of the framework join to a sleeve of 4 inches galvanized-iron pipe 9 inches long, which forms the bearing for the axle, made of 3-inch galvanized iron pipe. The gate is counterbalanced and operated by a connection to a system of two levers with a concrete block counterweight and a floating tank. The levers are connected together at one end or pivoting point and operate on two fulcrums; the gate is hung to the upstream end of one of the levers, the counterweight is placed near the pivoting point or connection point of the two levers, and the float is hung to the upstream end of the second lever. The float is a galvanized-iron tank, which fits into a concrete circular well built in the bank on one side of the structure. The well is connected at the bottom to the water on the upstream side of the check gate by an inlet pipe 4 inches in diameter with a pivot joint formed by two elbows, which permit

the raising or lowering of the upper end of the pipe. A similar connection is made by means of 2-inch outlet pipe with the water on the downstream side. The automatic regulation is obtained by the adjustment of the inlet and outlet pipes. The upper end of the inlet pipe is raised up to nearly the desired water level in the canal; the outlet pipe is adjusted so as to

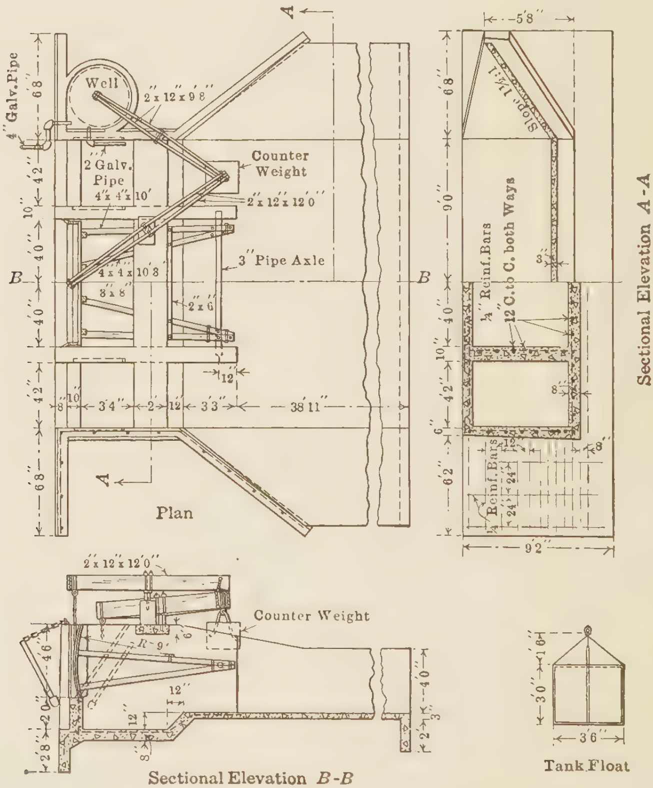


FIG. 121.—Automatic undershot radial gate. Turlock Irrigation District, Calif.

carry out of the well the water entering it through the inlet pipe with the gate held at the correct position. A raise in the water level, produced by an increased flow in the canal, will increase the flow entering the well; this will increase the depth of water in the well, and the greater flotation force acting through the levers will raise the gate about sufficiently to permit the excess flow to go through without a material variation in the water

level of the canal. A lowering in the water level, due to a decrease in the canal flow, will produce the reverse action. The gate mechanism and principle of operation is patented.

The use of these gates has given good regulation; the water level has been maintained near the full supply depth and this has prevented much of the trouble and breaks resulting from the burrowing of gophers. The greater depth maintained in the canal has a tendency to increase the seepage loss, but on the other hand the seepage loss through the holes made by burrowing animals has been practically eliminated.

The use of these automatic gates requires a certain difference in elevation between the upstream and downstream water level to operate the regulating system of levers and float, which may not be available with canals on flat grades having no excess fall. A difference in elevation of no less than about 12 inches is necessary.

REFERENCES FOR CHAPTER IX

A Special Type of Headgate for Irrigation Canals (Minidoka project)—p. 309 Eng. Record—March 12, 1910.

Rouse, H. M.—Cost of Constructing a Reinforced Concrete Check and Delivery Structure for an Irrigation Canal, p. 263 Eng. Contr.—Sept. 6, 1911.

CHAPTER X

LATERAL HEADGATES AND DELIVERY GATES

Object of and Types.—Lateral headgates are structures used at the head of main laterals, sub-laterals and distributaries to control and regulate the water entering them. Delivery gates are the structures placed at the points of delivery to the irrigator. Other terms commonly used to denote the same types of structures are: lateral turnouts and farm turnouts; also lateral takeouts and farm takeouts.

These structures are placed through the banks of the canal from which the supply is taken, and may be separate structures, or may be built as composite structures, usually with a check gate, in which case they may consist of a check gate with an adjacent single lateral or delivery gate on the upstream side, through one of the banks, or of two adjacent lateral gates, one through each bank. The check gate and the lateral headgates may also be formed to serve as drop structures. When the composite structure divides the flow between two or more laterals or distributaries of about equal size, so that no part of the structure can be properly called the check gate, then the terms sometimes used are bifurcation gates and division gates.

The term *bifurcation gates* is more properly applied to a structure which divides the flow between two main laterals, and the term *division gates* to a structure which divides the flow between two or more smaller laterals or distributaries.

The differentiation between the types of structures included in those terms is largely dependent on the required size and capacity; for instance, on projects where large irrigating heads are delivered, the corresponding delivery gate structure will have as large or even larger capacity than the distributary headgate structure used on projects where different irrigation practice is obtained. In the same way a main lateral headgate structure on one project may be smaller than a small lateral headgate on another project. There are, however, certain general principles of design which will apply to all sizes of structures, and these are con-

sidered below for the two distinct types of structures: the open channel type of headgate and the culvert type of headgate. In general a lateral headgate structure will have certain parts similar to those of the main headgate structure or regulator at the head of the diversion canal. An important difference is that the main regulator is placed on the river bank where it is subject to great fluctuations in water levels and to large hydrostatic pressure produced by flood flows, while the maximum hydrostatic pressure on the face of the lateral headgate is only that due to the full depth of water in the canal, which makes it possible in the open channel type of lateral headgate to close the entire waterway up to the full water supply level with gates without the panel wall, generally required in the main regulator structure. An open type lateral headgate is also similar in form and make-up to a check-gate structure, but will usually differ from it in the operation of the gates and in some cases in the form of the inlet and outlet. The principles and details of design presented in the discussion of the main headgates and especially of the check gates are therefore applicable to the design of lateral headgates.

OPEN TYPE OF LATERAL HEADGATE STRUCTURE

General Form and Parts.—This type consists essentially of a rectangular channel or short flume section with suitable inlet and outlet wings, built in an open cut through the canal bank. The rectangular channel is formed of a floor and two side walls, and for a wide structure the waterway may be divided into two or more bays, by frames, piers, or buttresses, on top of which is usually a foot-walk or platform for the operation of the gates, and in some cases a bridge floor or slab for road crossing, ranging in width from 10 to 12 feet wide when used occasionally, such as when on private roadway, and 14 to 16 feet or more when used on a public roadway. This combination makes the structure essentially of the culvert type.

In both the open channel type and the culvert type various forms are given to the inlet and outlet, depending on the position of the wings. The usual forms for the inlet are: (1) Wings at right angles to the side walls. (2) Parallel wings usually in the same plane as the side walls. (3) Flaring wings, forming an angle with the side walls of the structure not greater than

30°. When right-angle wings are used the entrance is set well back in the bank, and the approach is formed by shaping the earth to its natural slope of repose; this will usually require a length of wing wall equal to 1½ or 2 times the full depth of water above the floor of structure. When parallel wings or flaring wings are used, they extend out from their connection with the side walls, and their upper edges are sloped down to be below the surface. At their junction with the side walls, cut-off walls forming a collar around the structures are often used to give the desired length of path of percolation around the structure. Right-angle wings at the inlet do not form an entrance as well shaped as the parallel or flaring wings, but are more economical.

The outlet may be formed as the inlet, or may connect with the earth canal with warped surfaces. Right-angle wings with riprap, paving, or lining of the adjacent canal section are very commonly used.

Undershot and Overpour Gates.—The structure may be either overpour or undershot, depending on the method of operating the gates. The advantages and disadvantages of the two types of gates have been presented in the discussion of check-gates. The overpour type was shown to be preferable for a check-gate structure, because with variations in the canal flow upstream from the structure a more nearly constant water level could be maintained in the canal. The undershot type is preferable for a headgate structure, because any fluctuations in the water level of the supply canal will produce a smaller variation in the discharge through the gate. It is therefore important that check gates be of the overpour type and lateral headgates of the undershot type. Variations to this rule are sometimes made with certain special types of structures and when silt problems must be considered. For instance, when the water in the supply canal carries much silt and, because of the flat grade of a certain lateral, it is desired to keep the silt out of this lateral, then an overpour type headgate will be desirable. In the following discussion the undershot type is specially considered.

Hydraulic Computations.—The main dimensions of the structure will depend on the required capacity and the net pressure head which produces the flow through the opening of the undershot gate. The required capacity will be that of the lateral. The available difference in elevation between the water levels of the supply canal and of the lateral will only in

exceptional cases be sufficient to give the conditions necessary for free discharge through the gate opening. The opening will therefore usually be submerged with a difference in water levels, determined by the general topographic conditions and the form and position of the canal cross sections with respect to the ground surface. In comparatively level country less than 1 foot difference in elevation between the full supply water levels may be obtainable, but at least 1 foot and preferably 18 inches is desirable. When there is ample difference in elevation, the structure must be placed sufficiently low and made of such dimensions that with partial or low flow in the supply canal it will have the required capacity with minimum checking. A high velocity through the structure while decreasing the required dimensions is not desirable, at least for large structures, on account of the protection work necessary at the outlet. The rate of discharge per square foot of gate opening will usually not exceed 2 to 3 cubic feet per second, where little head is available, and 3 to 4 cubic feet per second where considerable head is available. The values of the differences in elevation required to produce the corresponding discharges are obtained from the formula:

$$Q = AC\sqrt{2\,gh}$$

where h is the head in feet represented by the difference in water levels.

C = a coefficient of discharge whose value will vary with the size of the opening, the form of the structure and the conditions of flow, but will usually range between 0.7 and 0.8.

Special values of C are given in Chapter XIII and in Vol. II, Chapter X.

Using the above formula and a coefficient of discharge of 0.7, the heads required are:

1.52 inches for a flow of 2 cubic feet per second per square foot of gate opening.
3.43 inches for a flow of 3 cubic feet per second per square foot of gate opening.
6.10 inches for a flow of 4 cubic feet per second per square foot of gate opening.

These minimum heads represent the desirable minimum difference in elevation between the inlet water level, corresponding to the low water flow at which the supply canal may be operated, and the outlet water level, corresponding to a full flow in the lat-

eral or distributary. These are the most unfavorable conditions of operations, and if the above desirable requirements can be obtained, then when the supply canal is operated at full flow or its water level checked up to full depth, the gate openings are regulated to correspond with the increase in velocity and, because of the greater head, closer regulation of flow through the structure is obtained. A surplus in the difference in elevation can be used to advantage for the installation of a measuring device, especially in the case of delivery gates.

Position of Structure.—The structure must be placed with the gate opening at least below the low water operating level in the supply canal and it will usually be desirable that it be submerged at the outlet to decrease exit velocity and to produce more favorable conditions for the measurement of water. To obtain this last requirement it is necessary to place the entire gate opening below the low water level in the lateral or distributary to which water is delivered. A common assumption is that a lateral or distributary will usually not be operated with a smaller flow than ½ or ⅓ of its full flow. The floor of the structure is commonly placed on the same level or a little lower than the bed of the supply canal, but in certain cases, such as when a distributary is supplied from a relatively large deep canal, it may be more economical and equally satisfactory to place the floor of the structure nearer the top of the bank.

Dimensions of Gate Openings and Gates.—The height of the gate opening is determined, as indicated above, by the position of the floor and the requirement for submergence. The total width is then obtained from the known area and the height, and may be divided into a number of bays separated by piers or buttresses each regulated with a gate. The gates most commonly used are vertical rectangular gates of wood or steel usually not over 4 to 6 feet wide, operated when of large size by either a lever or a screw lift or rack and pinion lifting device. The rack and pinion device, on account of its rapid action and its greater efficiency, is well adapted. The use of radial Taintor gates has been adopted for the larger headgate structures on a number of projects. They are specially well suited for this purpose on account of the east of operation, the undershot discharge and the large width of gate opening which they permit. The use of Taintor gates in the place of rectangular gates will reduce the number of piers and lifting devices and will do away with the

upper part of the piers and side wall usually necessary for the gate frames and lifting devices of rectangular gates.

Provisions to Prevent Undermining or Washing Around the Structure.—The safety of the structure against these actions will depend largely on the care taken in backfilling the structure, as well as on the length of the path of creep under or around the structure. To obtain the required length of the path of creep, cut-off toe walls and wing walls must extend well into the bed and banks. A large number of structures which have proven to be stable indicate the following rule:

First.—Make the length of floor measured downstream from the gate opening to the outlet end equal to not less than 2 or 3 times the height of gate; the greater length being used for easily eroded soil.

Second.—Make the path of percolation around or under the structure equal to 4 to 5 times the full depth of water on the floor of structure at the inlet for clay loam soils, and from 6 to 8 times for average loamy soils and sandy soils.

Third.—Make the depth of cut-off toe wall at the inlet to the structure equal to the full depth of water on the floor when in sandy soils and ½ this depth when in clay soils. The depth of outlet cut-off wall, except in very porous soil easily eroded, can usually be made less than that of the inlet toe wall.

Examples of Open Channel Type of Headgate.—A bifurcation headgate structure for the division of the flow between two main laterals on the Twin Falls Salmon River Water Co., Idaho, is shown in Fig. 122. The capacity of the larger lateral is 175 second-feet, and of the smaller 75 second-feet. The flow in each lateral is regulated by Taintor gates. The control by undershot gates is in this case desirable, because any variation in the flow reaching this point of division should usually be apportioned about equally between the two laterals. If, however, the flow in the lateral was a relatively small fraction of the entire flow, then the structure across the main lateral would be essentially a check gate and should preferably be of the overpour type. On this project the division of flow between main laterals has in general been made with the same type of bifurcation works (Plate XVI, Fig. A).

The type of headgate used on the South San Joaquin irrigation system, California, is shown in Fig. 123. This structure is designed for a lateral capacity of about 75 cubic feet per second

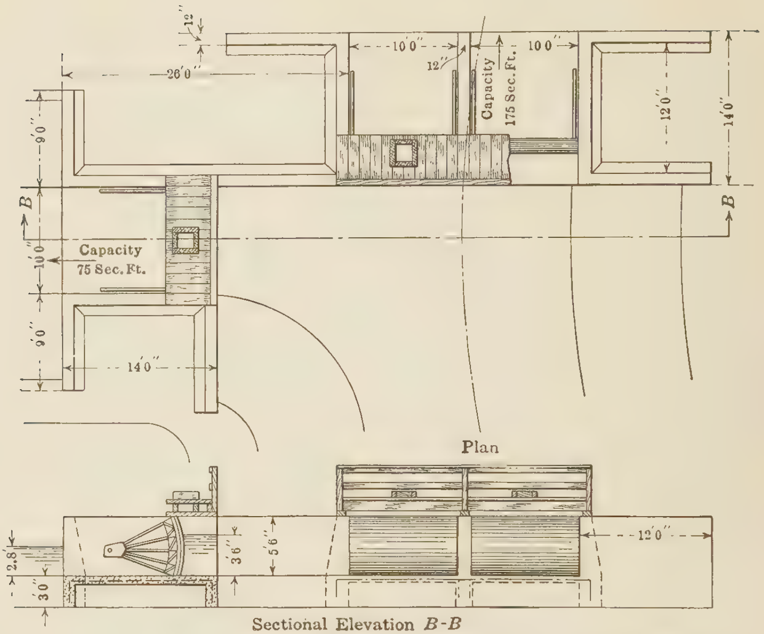


Fig. 122.—Division gates for two main laterals. Twin Falls Salmon River Water Co., Idaho.

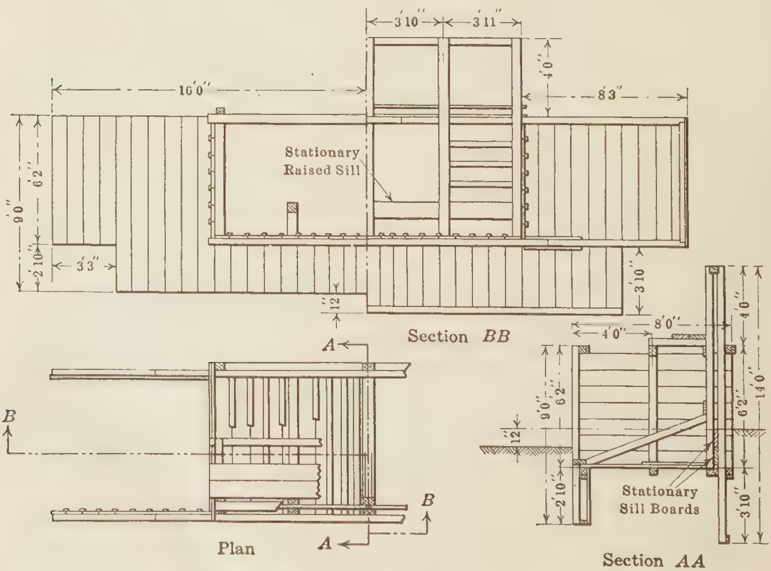


Fig. 123.—Lateral headgate with drop. South San Joaquin Irrigation District, Calif.

FIG. A.—Bifurcation works on main canal and lateral of Twin Falls Salmon River Land & Water Co., Idaho.

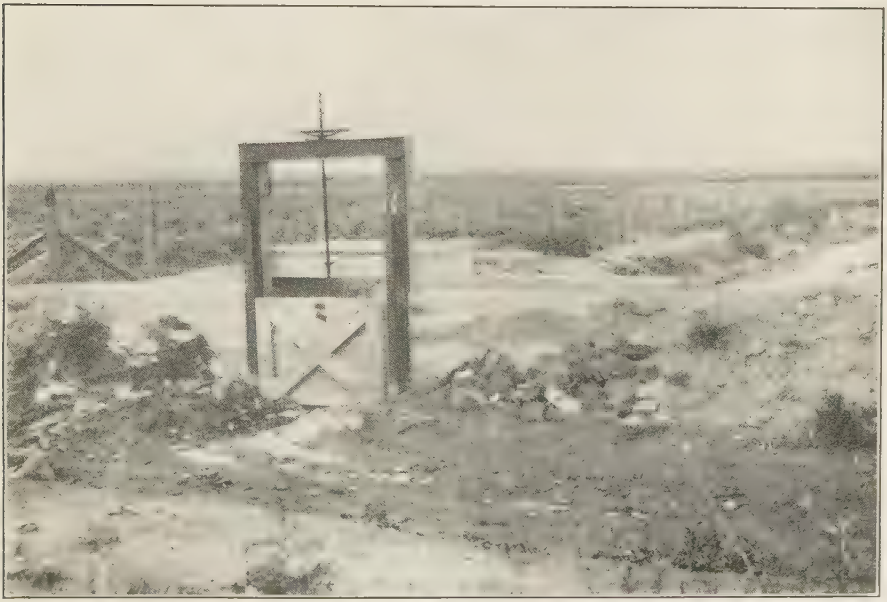


FIG. B.—Lateral headgate. Twin Falls Salmon River Land & Water Co., Idaho.

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Plate XVI.

Fig. C—Diversion gates and drops for small laterals. Umatilla Project, Ore.

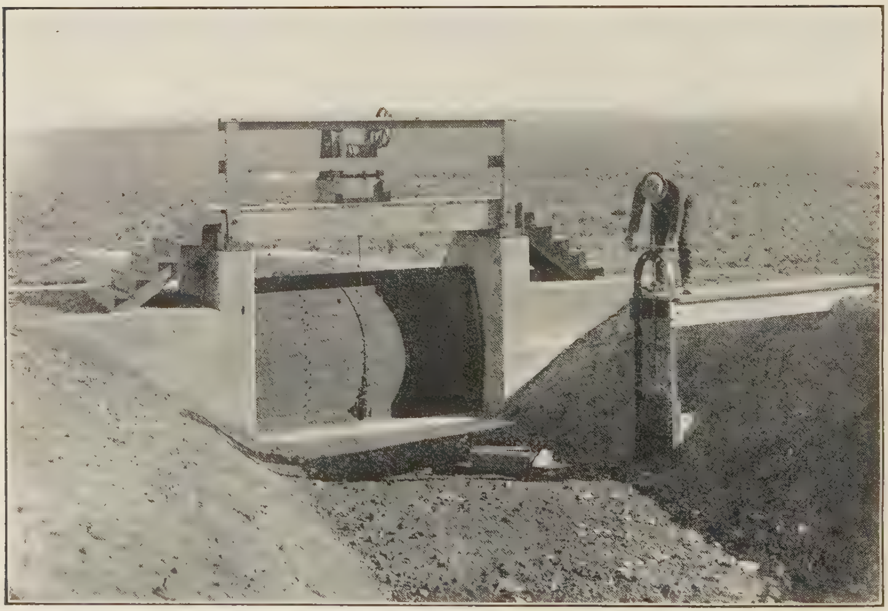


Fig. D.—Taintor check gate and pipe delivery gate used by Twin Falls Salmon River Land & Water Co., Idaho.

and combines with it a drop in canal grade of 1 foot. The structure is entirely built of wood with substantial wings and cut-off walls at the inlet and outlet. The gate openings are separated by wooden frames and are regulated by screw lift wooden gates. The floor of the structure is placed about 8 inches below the bed of the lateral, which insures this minimum depth of water cushion.

Plate XVI, Fig. B, shows an open channel type of small lateral

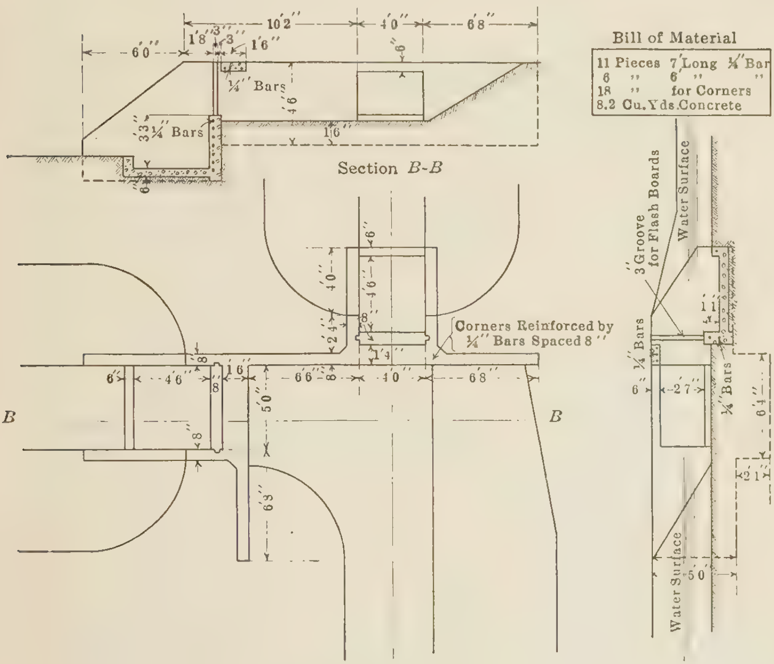


FIG. 124.—Division gates and drops for small laterals on Umatilla Project, Ore.

headgate used by the Twin Falls Salmon River Water Co. of Idaho.

Fig. 124 and Plate XVI, Fig. C, show division gates for two small laterals, each carrying about 20 to 30 cubic feet per second. The structure forms two openings, both regulated by flashboards, and serve also the purpose of drops on both laterals.

Fig. 125 shows a simple type of delivery gate on the Orland project, California. The irrigating head delivered to farmers ranges from about 5 to 12 second-feet and this structure may

be used to deliver 15 to 20 second-feet. The total and average cost of construction for 395 structures of this type, containing 477 cubic yards of concrete, is as follows:

Cost of 395 Delivery Gates on Orland Project, California

Classification	Total cost	Cost per structure
Superintendence	$272.00	$0.69
Equipment depreciation	70.87	0.18
Design	12.50	0.03
Miscellaneous supplies	103.90	0.26
Labor: Excavation	1,338.18	3.39
Building forms and placing reinforcement	1,265.72	3.20
Mixing and placing concrete	1,185.21	3.00
Finishing	6.75	0.02
Hauling sand and gravel	872.01	2.21
Hauling water	111.83	0.28
Miscellaneous	390.07	0.99
Backfilling	451.33	1.14
Hauling material	577.44	1.46
Cement	1,095.21	2.77
Reinforcement	263.54	0.67
Lumber for forms	84.11	0.21
Miscellaneous material—guides, bolts, etc.	1,679.31	4.25
Corral	152.68	0.39
Total labor and material cost	9,932.66	25.14
General expense	1,617.97	4.10
U. S. Engineering expense	527.53	1.34
Total cost	$12,078.16	$30.58

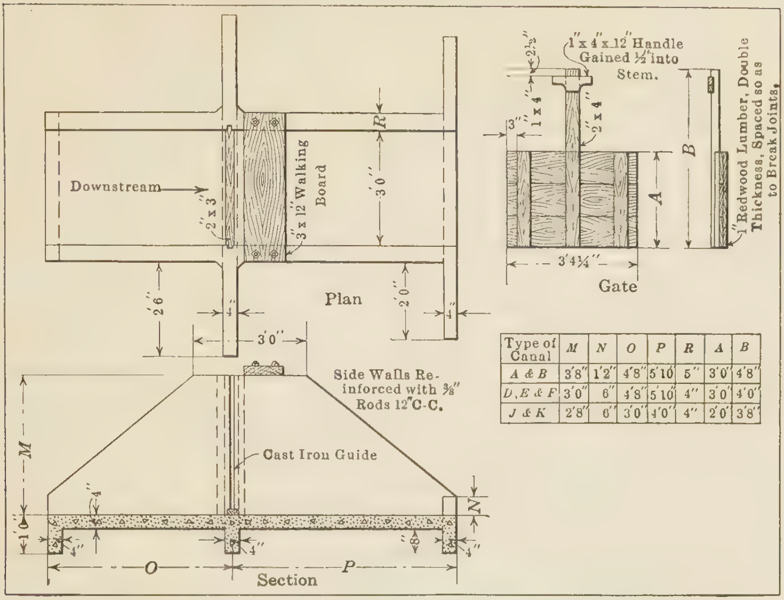
Labor cost as follows: Foreman $100 per month, 2-horse teams $4.50 per day; carpenters $3.50 per day; laborers $2.40 and $2.60 per day.

Unit cost of materials as follows: Cement $2.20 per barrel; reinforcement $2.50–$2.65 per c.w.t.; lumber $13.00 per M.B.M.; cast-iron guides $1.55–$1.80 each.

Average haul: 2½ miles for gravel; 2¾ miles for cement and supplies.

A similar type of wooden delivery gate used also for approximate measurement on the system of the Yolo Water & Power Co., California, is presented in the chapter on Measurement of Water.

A combination of small distributary check gate with delivery gate used by the Sacramento Valley Irrigation Co., California, is shown in Fig. 126. Two forms of construction are shown



Type of Canal	M	N	O	P	R	A	B
A & B	3'8"	1'2"	4'8"	5'10"	5"	3'0"	4'8"
D, E & F	3'0"	6"	4'8"	5'10"	4"	3'0"	4'0"
J & K	2'8"	6"	3'0"	4'0"	4"	2'0"	3'8"

FIG. 125.—Standard delivery gate. Orland Project, Calif.

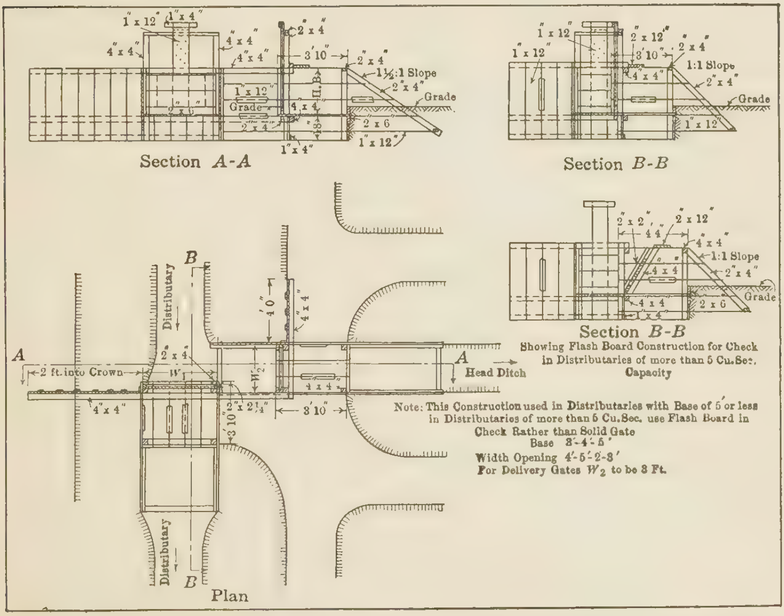


FIG. 126.—Small distributary check gate and delivery gate. Sacramento Valley Irrigation Co., Calif.

differing in the type of check gate. When the distributary is less than 5 feet wide at the base or carries less than 5 second-feet the regulating gate in the check is a straight lift undershot gate and the combined structure is essentially a division box. On larger distributaries sloping flashboards are used on the checkgate.

Culvert Type of Lateral Headgate.—Small structures of this type consist of a single short pipe tube, or of a short rectangular culvert through the bank of the supply canal, with suitable inlet and outlet and a slide gate at the inlet to regulate the flow. Larger structures may consist of two or more tubes or of a rectangular culvert divided into two or more compartments. In the smaller structures the inlet may be formed by surrounding the end of the tube with a concrete block, with its upstream face shaped to the canal side slope. Around this block may be placed riprap or concrete lining, extending on both sides and up to near the top of the bank. To prevent settling and cracking of this lining, by the water washing under it or by the settlement of the earth backing, it is desirable to extend the lower end below the canal grade and to provide along the two side edges of the slab cut-off walls or ribs extending well into the bank to firm material. Another form for a pipe inlet consists in placing the gate frame vertically, in which case it stands away from the bank and requires a foot-walk reaching from the top of the bank to the lifting device (Fig. 127). A number of manufacturers in the West make gates and gate frames specially designed for these types of lateral or delivery gates.

Where the above forms of structure are not used, and especially for larger structures, the face of the entrance to the tube or culvert is placed back in the bank to be about in the same vertical plane as the top edge of the water surface. The approach to the opening can then be made in front of right-angled wings by shaping the earth to its natural slope of repose (Fig. 131) or by using parallel or flaring wings in the same manner as for the open type. Placing the entrance to the structure back in the bank forms a basin in front of the gates, in which the comparatively still water when the gates are closed favors the disposition of sediment; however, no trouble need be expected unless the water carries much silt and the gates are closed for sufficient long intervals to permit the packing of the silt against the gates. The choice

between the types described above will depend largely on the comparative cost.

The hydraulic computations and the factors determining the position of the tube or culvert box in the bank are essentially the same as those discussed in the open channel type of structure. To utilize the full cross-section of the tube or culvert and to produce conditions favorable to the measurement of water, the tube or culvert is usually placed to produce submerged discharge for all stages of water levels at the inlet and outlet. When these water levels have been determined, the area of the cross-section

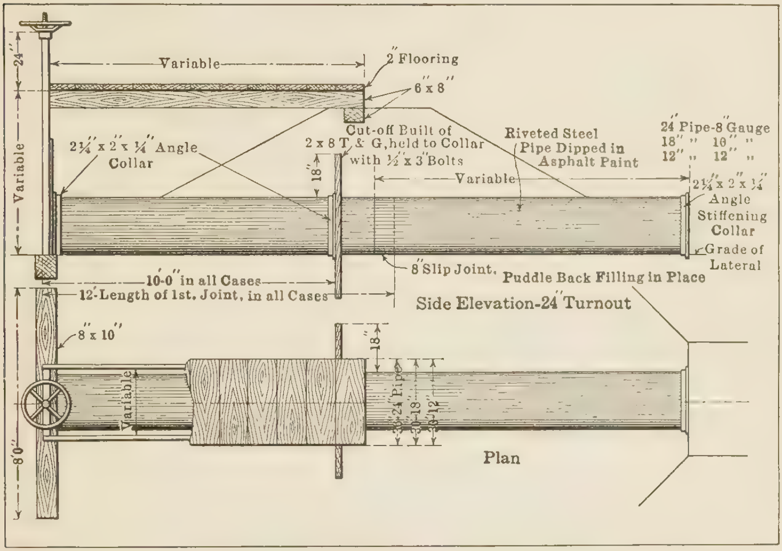


FIG. 127.—Pipe delivery gate. Twin Falls Salmon River Water Co., Idaho.

of the tube or culvert is determined from the formula $Q = CA\sqrt{2gh}$ or from computations of the entrance and exit loss of head and the loss of head due to the frictional resistance in the tube or culvert. The above formula is generally used with values for the coefficient of discharge C, which depend on the form of the inlet and outlet and the length of the tube. The values given in Vol. II, Chapter X, for short tubes and short pipes may be used for purposes of computation. Special values for small wooden box culvert delivery gates are given in the chapter on Measurement of Water (Chapter XIII). After the determination of the cross-sectional area the number of tubes or compartments and the

dimensions of the cross section must be selected. The choice between one or more pipes, or between a culvert of one or more compartments, must be based largely on a cost comparison, in which the cost of the gates and lifting devices may be an important factor. The bottom of the pipe or floor of the culvert

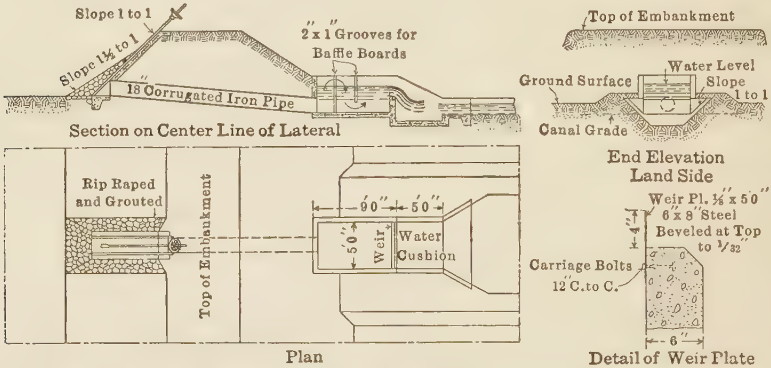


FIG. 128.—Small distributary headgate and measuring weir used by Southern Alberta Land Co., Canada.

must usually be placed at about the same level or lower than the bed of the canal, but seldom lower than about 1 foot below the bed in order not to produce an excessive earth pressure on the culvert or pipe and not make the height of inlet and outlet walls too great.

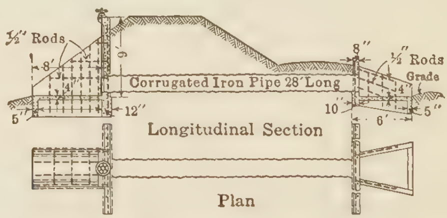


FIG. 129.—Small distributary headgate used by the Sacramento Valley Irrigation Co., Calif.

Fig. 127 and Plate XVI, Fig. D, show a standard type of pipe delivery gate used by the Twin Falls Salmon River Water Co., Idaho. These are made of riveted steel pipe, either 12, 18 or 24 inches in diameter, with a cast-iron screw lift gate at the inlet. Smaller delivery gates are made of 6, 8 or 10-inch machine-banded wood pipe with a wooden straight lift gate at the inlet.

Fig. 128 shows the type of pipe distributary headgate with

measuring weir used for distributaries of 15 to 20 cubic feet per second carrying capacity on the irrigation system of the Southern Alberta Land Co. in Canada. The sloping wall at the inlet is a reinforced concrete slab placed inside the bank of the canal with grouted riprap on the sides and at the bottom. This same type is used for larger structures on a number of projects; it is then desirable to add cut-off walls along the side edges and bottom of the slab extending well into solid material to give a solid foundation and prevent undermining. The overpour wall of the weir box should in the case of silty water have a bottom opening to sluice out deposited material.

The form of distributary headgate used by the Sacramento Valley Irrigation Co. in California is shown in Fig. 129.

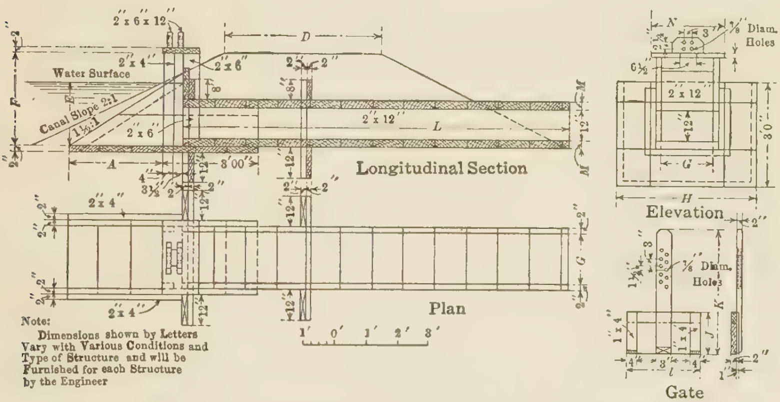


FIG. 130.—Common type of wooden delivery gate.

A form of wooden delivery gate quite generally used on many projects, especially in Montana, Wyoming, Utah, Idaho and Washington, is illustrated by Fig. 130. The common sizes of the box culvert opening range from 12 × 12 inches to 12 × 36 inches. For the larger sizes and in material easily eroded a wooden wall built at the outlet end of the box is desirable. This type of structure, made if necessary in larger sizes, is also commonly used for the headgates to distributaries. The smaller sizes can often be more economically built at a central point or lumber yard, then hauled and installed. On the Oakley project, Idaho, with lumber at $25 per M.B.M., the costs of 12 × 24-inch box delivery gates of different lengths, at the yard, were as follows: For 12 × 24-inch boxes 6 feet long, $8.10;

8 feet long, \$8.50; 10 feet long, \$9.10. The total cost and unit cost of 159 box delivery gates installed on the Boise project, Idaho, were as follows:

COST OF 159 DELIVERY GATES, BOISE PROJECT, IDAHO.

	Total cost	Unit cost per M.B.M.
Hauling material	\$244.70	\$7.15
Excavation and backfill 1,677 cubic yards	806.64	23.59
Installing structures	419.45	12.26
Engineering and supervision	136.82	4.00
34,204 feet of lumber at \$20.25 M.B.M.	692.63	20.25
Nails and miscellaneous	81.04	2.37
Total	\$2,381.28	
Cost per M.B.M. of lumber		\$69.62
Cost per structure	\$14.98	

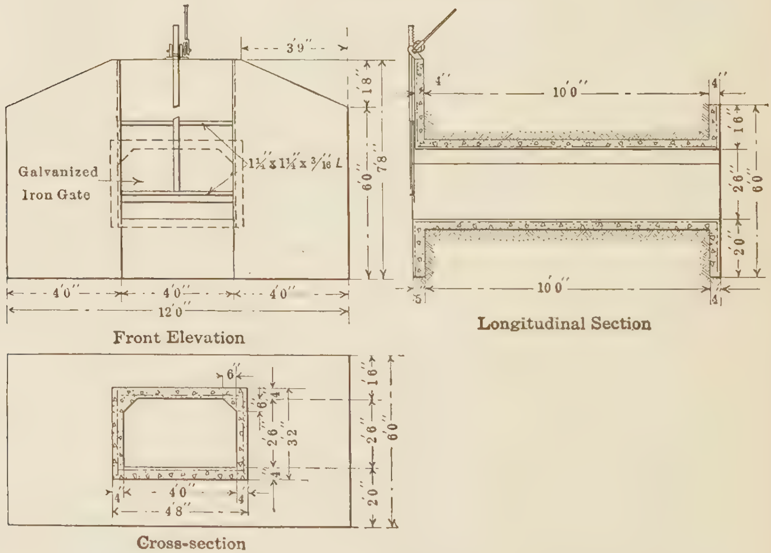


FIG. 131.—Delivery gate. Turlock Irrigation District, Calif.

A simple type of large reinforced concrete delivery gate used on the Turlock Irrigation District system is shown in Fig. 131. On this project irrigating heads of 15 to 25 cubic feet per second are commonly delivered. It can be used as a lateral or distributary headgate to deliver considerably larger volumes than a single irrigating head. The structure consists of the rectangular re-

inforced concrete culvert box with inlet and outlet end walls. The inlet must be set back in the canal bank, and the approach channel in front is formed with the natural slope of the earth. The gate is made of No. 14 gauge galvanized sheet iron, stiffened with horizontal angles near the bottom and along the top. The gate slides in grooves, between two metal plates, riveted to the gate frame with a spacer in between, and is operated with a rack and pinion lifting device. The structure contains 4.10 cubic yards of concrete; its cost as reported is as follows:

Gravel 6¾ tons	$17.86
Cement, 5 barrels	14.50
Sand, 1 cubic yard	0.85
Reinforcement, Clinton wire 3 × 12 inch mesh No. 6 and 10 wires 301 square feet	10.07
Gate in place	22.50
Forms and labor	23.40
Excavation, backfill	9.32
Miscellaneous	5.50
Insurance	1.90
	$105.90

The type of distributary or lateral headgate designed for the Sacramento Valley Irrigation Co., California, shown in Fig. 132, differs from that used on the Turlock irrigation system in the addition of inlet and outlet wings with floors in between. These form better shaped entrance and outlet, but increase considerably the cost of the structure. The arched roof and side walls of the culvert conduit are reinforced longitudinally with ½-inch rods, 12 inches center to center, held in place during construction by ½-inch transverse rods spaced 5 feet apart. The thin floor is reinforced with wire mesh. On a number of the Reclamation Service projects the same type of reinforced concrete lateral headgate has been used, with rectangular box culvert conduits and in some cases with flaring inlet wings. The outlet flaring wings are often omitted. An example of a double compartment lateral headgate of the same type is illustrated by the structure designed for the Flathead project, Montana, Fig. 133. This structure is designed for a capacity of 65 cubic feet per second. Cut-off walls are formed at the junction of the inlet wings with the culvert box to provide additional precaution against creepage of the water around the structure. The inlet of each compartment is regulated with a standard rectangu-

lar gate and a second set of grooves for flashboards is provided for emergency regulation. Between the outlet wall and a secondary downstream toe wall the canal section is pro-

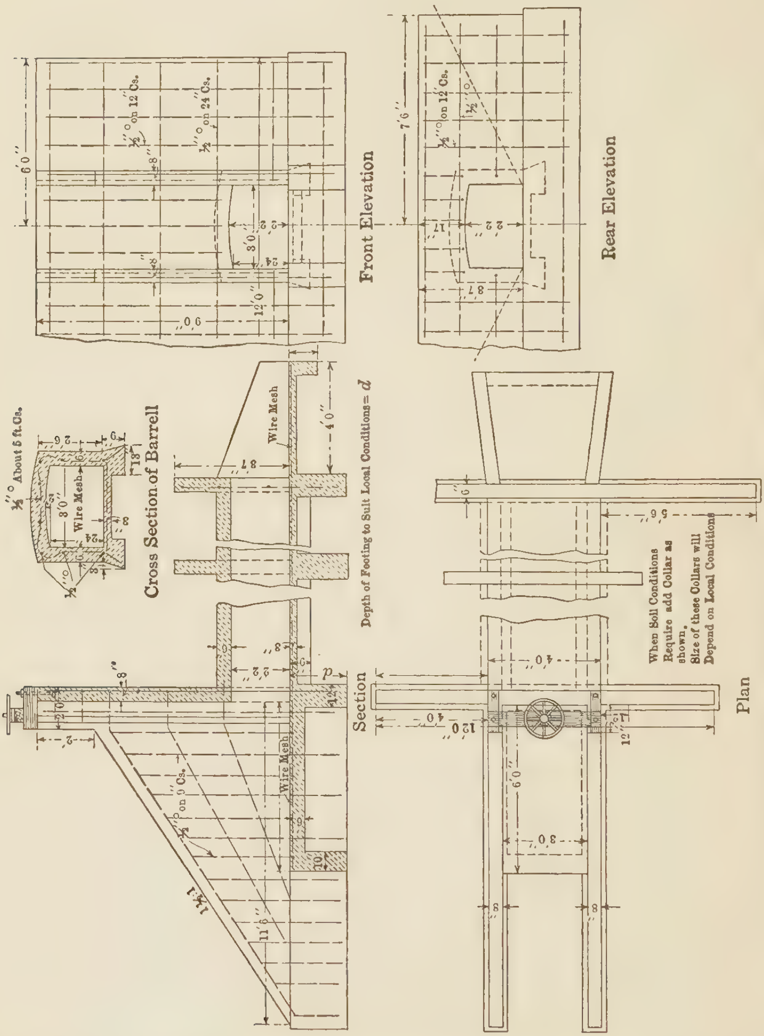


FIG. 132.—Small lateral headgate. Sacramento Valley Irrigation Co., Calif.

tected for a length of 10 feet with grouted paving and depressed about 18 inches below the canal grade. Provision is also made to increase the depth of this basin by raising the crest of the toe wall with flashboards. This stilling basin is desirable at the

outlet when the velocity through the culvert is high, and with the provision for flashboard makes it possible to raise the outlet

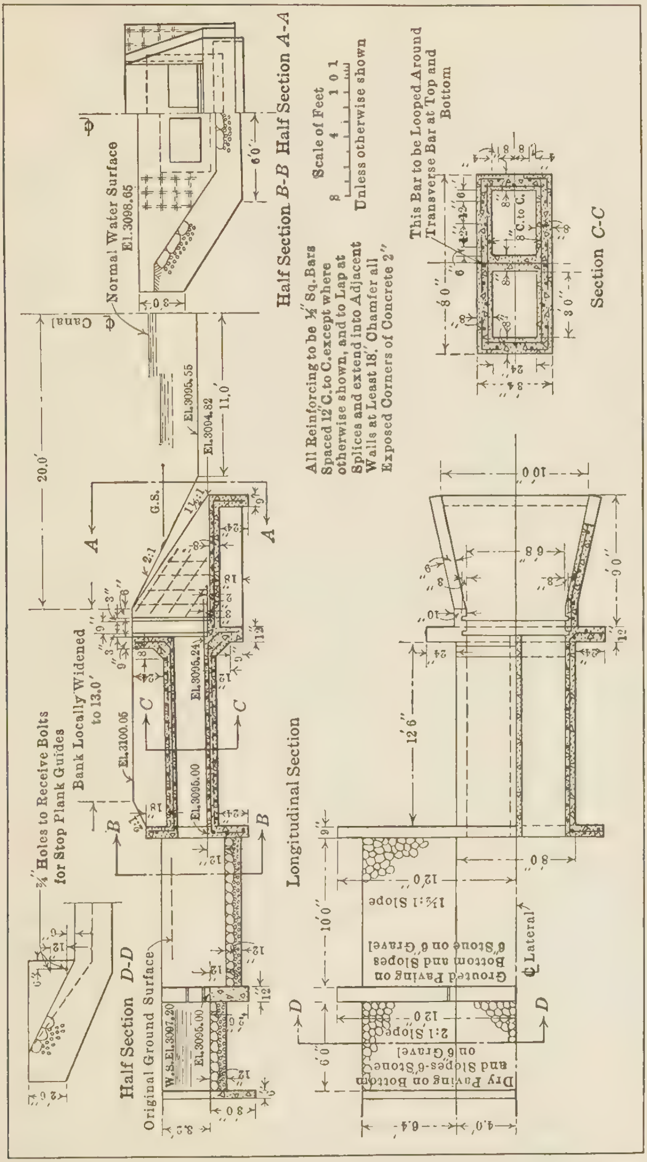


FIG. 133.—Double compartment lateral headgate. Flathead Project, Mont.

water level and obtain an approximate measurement of the flow by considering the structure as a tube with submerged outlet.

A good example of a large reinforced concrete lateral headgate is illustrated by Fig. 134. This structure, designed for a main lateral of the Salt River project, Arizona, is formed of three

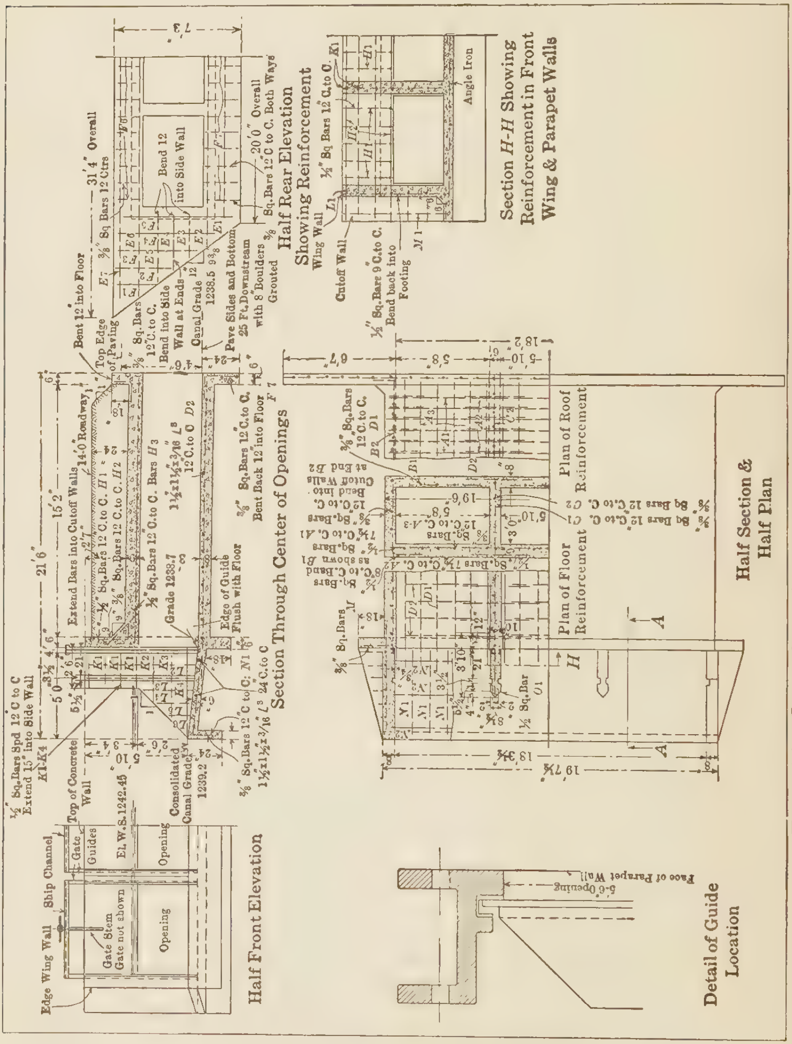


FIG. 134.—Large lateral headgate. Salt River Project, Ariz.

rectangular compartments, each regulated with a screw lift gate, sliding in metal guides, extended to support the cross channel on which the operating wheel operates.

CHAPTER XI

ROAD AND RAILROAD CROSSINGS WITH CANALS, CULVERTS, INVERTED SIPHONS AND BRIDGES

General Description and Types.—*A culvert* consists of a pipe, box or arch conduit placed under the roadway, with its floor or bottom at about the same elevation as the bed of the canal, and of inlet and outlet walls to connect the ends of the conduit with the sides and bottom of the canal.

An inverted siphon is essentially a culvert; it differs in that the conduit is placed deeper below the bed of the canal, which may require a different form of inlet and outlet structure.

Bridges are usually made either of timber or of reinforced concrete. The type of timber bridge most commonly used is the simple stringer type, with one or more intermediate piers for wide canals. Timber truss bridges are occasionally used for wide canals, when it is desirable to eliminate the use of piers. Reinforced concrete bridges, depending on the form of design, are commonly flat slab, T-beam, or girder bridges. Arched concrete bridges are used in a few cases, when sufficient clearance above the water surface can be obtained for the rise of the arch. Steel truss bridges have been used in very few cases and only for the crossing of very wide canals.

Selection of Type of Structure.—The selection depends on the relative positions of the canal and the roadway, on the capacity or size of the canal, on the traffic importance of the roadway, and on the cost.

When the crossing is made with a railway, the elevation and position of the tracks are fixed and cannot be adjusted to fit the structure; this will also apply in most cases to important highway roads. But on less important roads, such as cross roads or farm roads of light traffic, the elevation of the road surface may be raised to permit the use of a bridge instead of a culvert or siphon, and the alignment may be shifted by introducing curves to cross the canal at right angles. When the canal is small, a change in its alignment may be preferable to a change in the road alignment. On large canals skew bridges must often be used.

Where the full water supply of the canal is lower than the surface of the roadway, either a bridge or culvert may be used. This condition will usually be obtained when the canal is all or nearly all in cut, and the road surface is at or near the natural ground level. Where the canal is in deep cut, a bridge crossing at the ground level will require a comparatively long bridge, and for small canals at least the crossing may be more economically made with a culvert.

Where the full water supply level is higher than the surface of the roadway, an inverted siphon must be used. This condition will usually occur where the canal is partly in fill and the roadway is either at the ground surface or in cut.

The final selection will be determined from a comparison of first cost and ultimate cost The comparison must include a consideration of the loss in head through the different types and sizes of the structures, for which the formulas presented in Vol. II, Chapter X, may be referred to.

The following discussion and the examples selected present some of the general principles of proportioning and of hydraulic design and the suitability of each type. The details of structural design are excluded, as these can be obtained from some of the standard text-books on this subject.

Culverts and Inverted Siphons.—These may be classified according to the form of the conduit and the material of which it is built. The forms of conduit are: The circular form or pipe, the arched form, and the rectangular box.

The pipe culvert may be made of cement, vitrified clay, wood, reinforced concrete, steel, corrugated iron, or cast-iron pipes.

Cement pipe is usually the hand-tamped cement mortar pipe, made in sizes up to 36 inches in diameter. For culvert purpose it is seldom used for sizes greater than 24 inches and for sizes of 18 inches and above, it is desirable to strengthen the pipe with hoops of 1/8-inch wire, spaced about 6 inches apart. The pipe is used in highway crossings for the ordinary condition of highway traffic. The minimum depth of backfill on the roof of the pipe should be from 1 to 1 1/2 times the diameter and never less than 18 inches. The pipes must be carefully bedded and laid, to obtain a uniform distribution of the external pressure. The bottom of the trench must be well graded, and the backfill material thoroughly tamped up to at least 6 inches above the top of the pipe. The pipe must not lay on projecting rocks and the

backfill material must be selected or rocks removed. When in rocky ground or on hard material it is desirable to lay the lower part of the pipe on a bed of sand or gravel, and in cold climates a cover of sand or gravel all around the pipe at least 6 inches thick may be advantageous in providing drainage around the pipe and protection against the frost. When the pipe is subject to heavy loads or placed under deep fills, it may be desirable to increase its strength by placing a bed of lean concrete for the lower third of the pipe. The cost of the pipe may be subject to wide variations. Where the pipe can be purchased from a local pipe factory or where a sufficient quantity is required to justify the purchase of the equipment, it will be the cheapest kind of pipe.

Vitrified clay pipe is preferably the double strength sewer pipe, hard burned and salt glazed, from 12 to 36 inches diameter and made in lengths of 2 or 2½ feet. Its strength is usually greater than that of the ordinary hand-tamped cement pipe. The minimum depth of backfill and the care in bedding and laying is the same as for the cement pipe. The pipe must not be supported on the bell ends; to prevent this, grooves or depressions must be made in the bottom of the trench to receive the bell ends. The joints are made with 1 to 2 or 1 to 3 cement mortar. The cost of vitrified clay pipe will depend largely on the distance from the factory; it will usually be considerably more than that of cement pipe.

Wooden stave machine-banded pipe is made in sizes up to 24 inches in diameter, in even foot lengths from 8 to 20 feet. Its longitudinal stiffness is an advantage where subject to settlement, but on account of its shorter life under the conditions obtained in a culvert, it is not extensively used for culverts; however, because of its comparatively low cost it should be considered when selecting the material for an inverted siphon.

Reinforced concrete pipe may be cast or moulded sections, which are joined in the trench, or may be constructed monolithically in the trench. The cast sections are obtainable from pipe factories, or may be specially made on the project; they are usually made from 12 inches up to 6 feet in diameter and from 4 to 8 feet in length. The cast section pipe or monolithic pipe can be designed and constructed specially to resist the external pressure. Where a large quantity of pipe is to be made for culverts or pipe lines on the project, the pipe cast in sections can be made at a cost which will make it well adapted to highway crossings and

especially railway crossings, and more economical than the monolithic pipe. Under such favorable conditions it may be cheaper than vitrified clay pipe, at least for diameters above 18 inches. Monolithic pipe will usually cost more than reinforced concrete box culverts. A number of railroad companies make special concrete pipe for use at their crossings; for one of these companies reinforced concrete pipes, 24, 30 and 36 inches in diameter, in lengths of 12 feet and about 3 inches thick, are used.

Plain steel pipe is seldom used for culverts. To have sufficient stiffness its thickness must be several times that of corrugated pipe; therefore its cost is considerably greater. It should be protected against corrosion by a coating of asphalt or coal-tar, applied hot, usually by heating the pipe and dipping it in the mixture. A minimum depth of backfill of half the diameter is necessary. A minimum thickness of metal of $3/16$ of an inch is usually specified for diameters of 36 to 48 inches.

Corrugated pipe is made of relatively thin metal, in sizes ranging from 8 inches to 84 inches in diameter. For the smaller pipes 16 gauge metal is used; for pipes larger than 18 inches in diameter 14 gauge is used; and for pipes larger than 48 inches 12 gauge is used. To increase its resistance to corrosion, a number of manufacturers use a metal approaching wrought iron in composition, but made by a special process which removes practically all the impurities. This metal, commonly known as Ingot Iron, has been in use for less than 10 years, so that the claims of the manufacturers have not yet been demonstrated by actual experience. In the discussion of steel pipes for pipe lines (Vol. II, Chapter X), evidence is presented which indicates that under certain conditions, at least, its rust-resisting properties are not as great as generally assumed. The pipe has the advantage of stiffness and longitudinal strength, which makes it especially well adapted where a firm foundation is not obtainable or where settlement may be expected. Its relatively light weight is a great advantage in transportation and hauling. It may be used with a minimum depth of backfill of 6 inches, but 12 is preferable. The pipe must be placed in the trench with the longitudinal joints at the top and backfilled with material from which rocks have been removed. Thorough compacting of the material is required, at least for the lower half of the pipe. When the pipe is used as an inverted siphon or culvert depressed below the water level, it is necessary to make the joints tight by soldering. The Sacra-

mento Valley Irrigation Co. used a considerable amount of corrugated pipe for road culverts up to a canal capacity of about 10 cubic feet per second; the minimum depth of earth covering was 1 foot, and where there was more than 2 feet pressure head of water, it was found necessary to solder all the seams and joints.

Cast-iron pipe is obtainable in sizes from 3 inches to 84 inches in diameter; but, on account of its relatively greater thickness and cost in larger sizes, it is seldom used for culverts for sizes larger than 4 feet. The lighter weight of water pipe made in lengths of 12 feet is commonly used; for culvert purposes thinner pipes are made in lengths of 3 to 4 feet. Special culvert pipe is also made of three segments reinforced with projecting ribs on the outside. The minimum depth of backfill may be as small as 6 inches.

The selection of the kind of pipe to use will depend on the special properties favorable to certain local conditions and on the cost.

Adaptability of Kinds of Pipe.—Cement hand-tamped pipe and vitrified clay pipe should not be used at locations where there is danger of water freezing in the pipe; this may happen in regions of low winter temperatures, when the culvert is on a canal in which water may be carried or collected during the winter, or when the culvert pipe is placed below the canal bed in the form of an inverted siphon which cannot be emptied or drained out. Cement hand-tamped pipe and vitrified clay pipe should not be used in soils which become soft by the action of the seepage water from the canal and subject to unequal settlement; this is more likely to occur in clay soils and other retentive soils. These pipes, on account of their low cost, have been used to a considerable extent even on some of the irrigation projects in the northern states where winter temperatures are very low, but usually only in soils which drain readily and when the pipes will be empty during the freezing period. Cast-iron pipes are required by certain railroads, but on account of their high cost they are practically never used for highway crossings. Machine-banded wooden pipes and corrugated steel pipes have the advantage of longitudinal strength, which makes them specially desirable where the culvert pipe is subject to settlement. They are not liable to be damaged by frost and are therefore specially well adapted in cold climates for inverted siphons or culverts, which are not drained out in the winter. The wooden pipe will usually be cheaper, but is obtained only up to 24 inches in diame-

ter, and its life is usually considered shorter than that of corrugated pipe, although under certain favorable conditions, such as when placed in retentive soil, free from alkali and continuously wet or moist, well-coated machine-banded wood pipe has in some cases been found in good condition nearly 20 years after it was installed.

Reinforced concrete pipe has the advantage of durability and strength, but unless the project is near a manufacturing pipe plant will usually not be economical unless a sufficiently large quantity of the pipe is required for culvert and other purposes to justify the purchase of the equipment.

Comparative Cost of Culvert Pipes.—The approximate prices of the different kinds of pipes is given in the accompanying price list:

Diameter, in.	Cement hand-tamped	Vitrified clay	Corrugated iron	Wood-banded	Reinforced concrete	Cast-iron
10	$0.25	....	....	$0.25	$0.40	$1.20
12	0.35	$0.60	$0.90	0.30	0.50	1.40
14	0.40	....	1.10	0.42	0.60	1.80
16	0.45	....	....	0.50	0.70	2.10
18	0.55	1.10	1.30	0.56	0.80	2.60
20	0.65	1.50	....	0.62	0.90	3.00
24	0.90	2.00	1.70	0.75	1.20	4.00
30	1.25	3.20	2.50	....	1.75	5.00
36	1.65	....	3.00	....	2.50	6.70
48	....	....	4.25	....	3.50	....

The prices given are necessarily only of general value, and subject to considerable variations.

The prices for hand-tamped cement pipe include the cost of the pipe, a moderate haul of about 2 miles from the pipe yard, and the cost of placing the pipe in the trench and making the joints but not the cost of trenching.

The prices for vitrified clay pipe include the same items as for the cement pipe, but not railroad transportation.

The prices for corrugated iron and wood-banded pipe are the approximate factory price.

The prices for reinforced concrete pipe include the same cost items as the cement hand-tamped pipe, and are only obtainable for the conditions stated in the discussion of its use for culverts.

The prices for cast iron are taken at 2 cents per pound, which will be the approximate price delivered at the railroad station; it does not include hauling and laying.

Examples of Pipe Culverts.—Simple forms of corrugated pipe

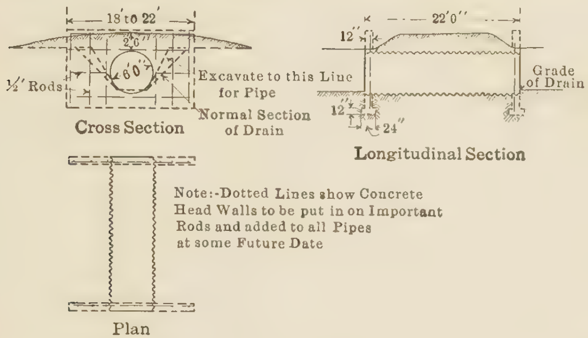


FIG. 135.—Corrugated-iron culvert on deep drain for road crossing. Sacramento Valley Irrigation Co., Calif.

and plain cement concrete pipe culverts for highway crossings used respectively on the Milk River project, Montana, and Orland project, California, are shown on page 169, Vol. I. On the

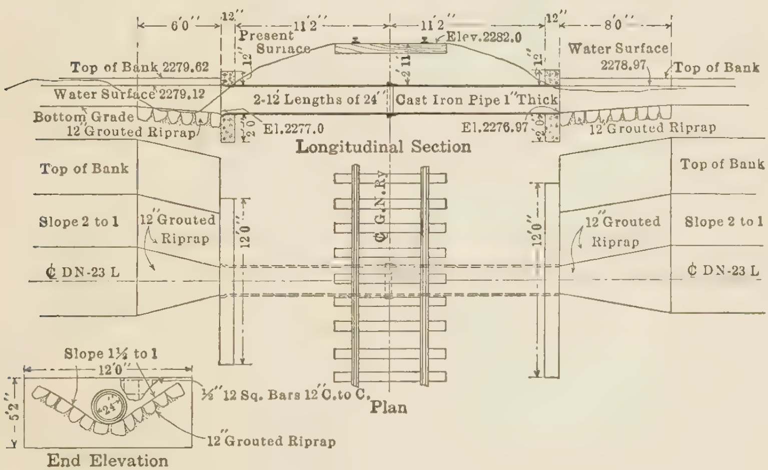


FIG. 136.—Cast-iron culvert under Great Northern Railway. Milk River Project, Mont.

Orland project the largest size concrete pipe used is 24 inches, for capacities up to about 15 second-feet; above this capacity the stringer bridge (page 171, Vol. I) is used.

The California State Highway Commission uses corrugated pipe culverts with plain end walls, from 12 up to 36 inches in diameter. For larger capacities box culverts and bridges are used. The Sacramento Valley Irrigation Co. in California used corrugated pipe for road crossings on distributaries up to a maximum required capacity of 10 second-feet. At these crossings the width of the roadway is at least 16 feet. The pipe is laid with its bottom a few inches below the bottom of the ditch with no inlet and outlet walls, and the pipe extended sufficiently to pass beyond the natural slopes of the fill. A minimum depth of 1 foot of covering is used. On a large deep drain on this project 6-foot corrugated pipe with end walls was used (Fig. 135). On the Milk River project small lateral culverts under the railway were formed of 24-inch cast-iron pipe, with simple end walls and grouted rip-rap inlets and outlets (Fig. 136).

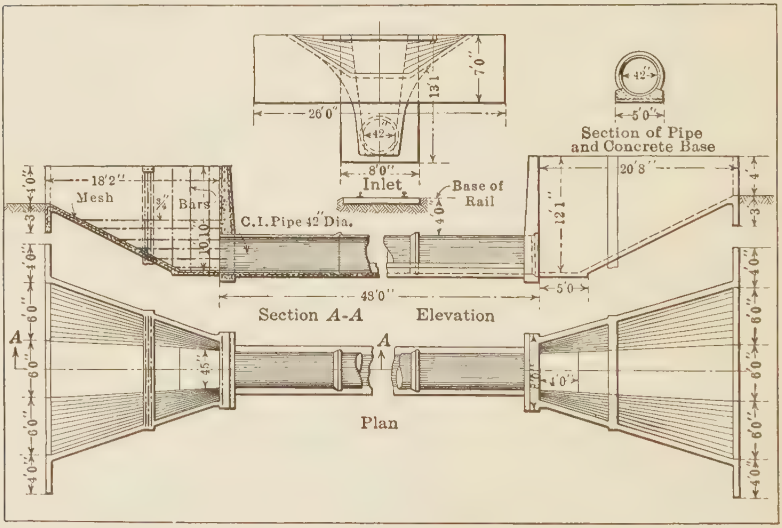


FIG. 137.—Culvert under railroad. Turlock, Calif.

Examples of Pipe Inverted Siphon Culverts.—These differ from the plain pipe culverts only in the pipe line being placed lower and in the form of inlet and outlet structure. The siphon culvert railroad crossing on the Turlock system, California (Fig. 137), consists of cast-iron pipe laid on a reinforced concrete bed, and of the inlet and outlet reinforced concrete structures. These two structures are practically the same; each is formed with warped wings.

The siphon culvert of the South San Joaquin Irrigation District (Fig. 138), used at a county road crossing, is formed of two lines of corrugated pipe with inlet and outlet structures, in which the side wing walls are formed with their lower part on a 2 to 1 slope to conform with the canal slopes and their upper part vertical. This form of wing is probably easier to construct than the warped wings.

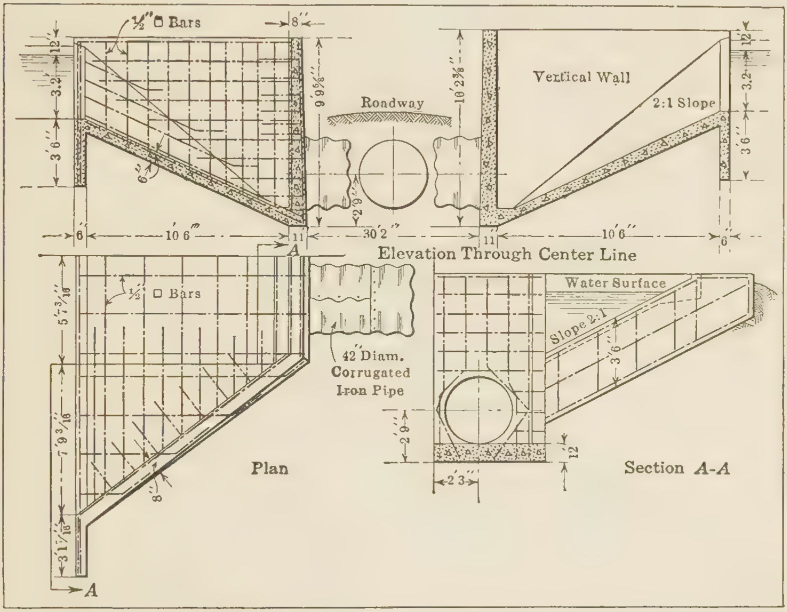


FIG. 138.—Inverted siphon under county road. South San Joaquin Irrigation System, Calif.

Box Culverts and Arched Culverts.—The conduit of the culvert may be a single compartment rectangular box, or it may be divided into two or more compartments separated by division walls. The form and dimensions will depend on the required capacity, the available head room, and the requirements for economic design. The conduit is usually made of wood or reinforced concrete. Wooden culverts will usually cost from 30 to 50 per cent. less than reinforced concrete culverts, but their useful life will depend on the kind of wood, the thickness of the timber planking, the strength of the framing, and the soil character and conditions.

Wooden culverts, of selected redwood, with planking not less than 2 inches thick, built in retentive soils, continuously moist, free from alkali, may be expected to have a useful life of 15 to 20 years; under less favorable conditions the useful life will probably be not over 8 to 10 years. Wooden culverts made of fir or pine have a useful life, ranging between 4 to 10 years. While not permanent, the lower cost of wooden culverts will justify their use in many cases for the smaller sizes.

In reinforced concrete box culverts the sides, roof and floor are reinforced concrete slabs, designed to resist the bending moment due to the external pressures. The cross-sectional area of the culvert is determined from the hydraulic computations. The proportioning of depth to width and the division of the

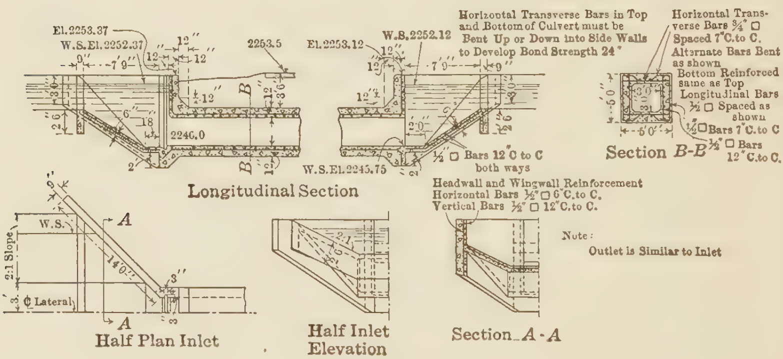


FIG. 139A.—Box culvert, railroad crossing. Milk River Project, Mont.

conduit into two or more compartments is determined largely from the requirements for economic design.

Arched culverts are usually built of plain concrete. Little or no economy is obtained from the use of reinforcement. The greater volume of concrete in an arched culvert will usually make the box culvert more economical.

Examples of Box Culverts.—A small-sized box culvert for railroad crossing of the inverted siphon type, used on the Milk River project, Montana, is illustrated by Fig. 139A. This structure is heavily reinforced.

A lighter design of a similar type of structure with simpler inlet and outlet structures, used on the Orland project, California, is shown by Fig. 139B. This structure is used for canal capacity of 30 cubic feet per second.

A large-sized box culvert for railroad crossing on the Umatilla project is shown by Fig. 140 and Plate XVII, Fig. A. The

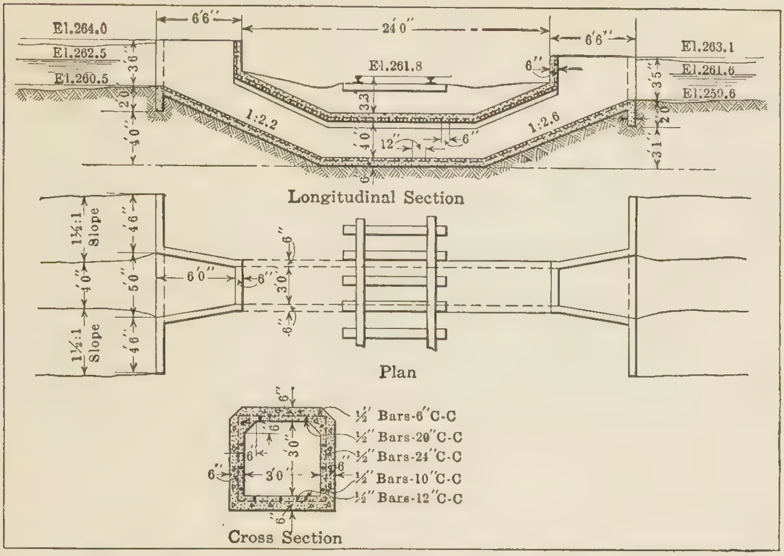


FIG. 139B.—Reinforced concrete box culvert for railway crossing. Orland Project, Calif.

structure is used on the main canal of 300 second-feet capacity. The inlet and outlet are formed with warped surfaces, consisting

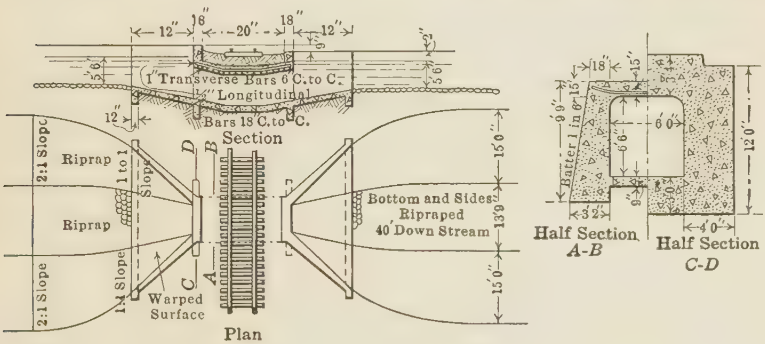


FIG. 140.—Railroad crossing. Umatilla Project, Ore.

of concrete wings and floor extended with 20 feet of riprap upstream and 40 feet of riprap downstream. The box of the culvert is formed of heavy plain concrete side walls, a heavily rein-

forced roof 15 inches thick, and a 9-inch floor. Small cracks have occurred at the junction of the wings with the box, probably because of the lack of reinforcement anchorage.

Large-sized box culverts are essentially the same as reinforced concrete slab bridges, of which examples are presented further. Additional examples of culverts are also shown in the discussion of culverts used at drainage crossings with canals.

Types of Bridges.—The types of bridges used for highway crossings are:

1. The wooden stringer bridge, which has a wooden plank floor nailed to stringers spanning the distance between abutments or piers.

2. The reinforced concrete flat slab bridge, which has a flat slab floor of uniform thickness, reinforced to carry the load over the span.

3. The reinforced concrete T-beam slab bridge, which has a relatively thin floor slab, supported on reinforced concrete beams, built monolithically with the slab and spanning the distance between supports.

4. The I-beam stringer bridge, similar to the T-beam type, with I-beams to carry the load over the span. The floor slab is supported on the I-beams, or it may be built to incase the I-beams in the concrete; a modification of this last form consists of arches formed between the I-beams by building the floor on arches of corrugated iron, supported on the lower flange of the I-beams, and spanning the width between them.

5. The reinforced concrete girder bridge has deep reinforced concrete beams or girders formed at each side of the bridge floor, acting also as railing or parapet walls and designed to carry across the span the total load transmitted by the floor, which may be either a flat slab, a T-beam slab, or an I-beam slab, supported by and built with these main girders.

6. The plain concrete and reinforced concrete arch bridges.

7. Wooden and steel truss bridges.

Wooden Stringer Bridges.—This type of bridge is that most extensively used on irrigation systems. For small canals it will be a single span, supported at each end on abutments (Plate XVII, Fig. B). Examples of this type are shown on pages 171 and 172, Vol. I. The maximum span may be determined from the maximum commercial length of stringers, and is usually not greater than 20 feet. For large canals the length of the bridge

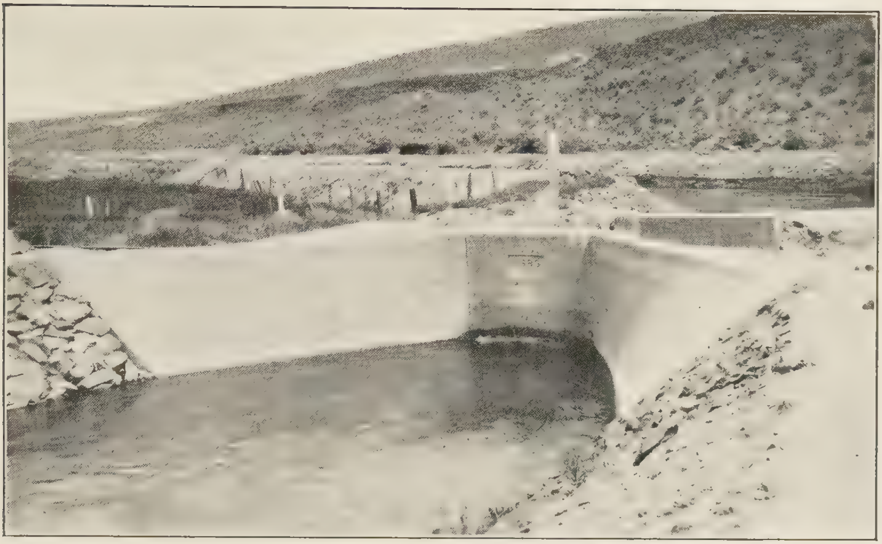


FIG. A.—Box culvert for railroad crossing. Umatilla Project, Ore.

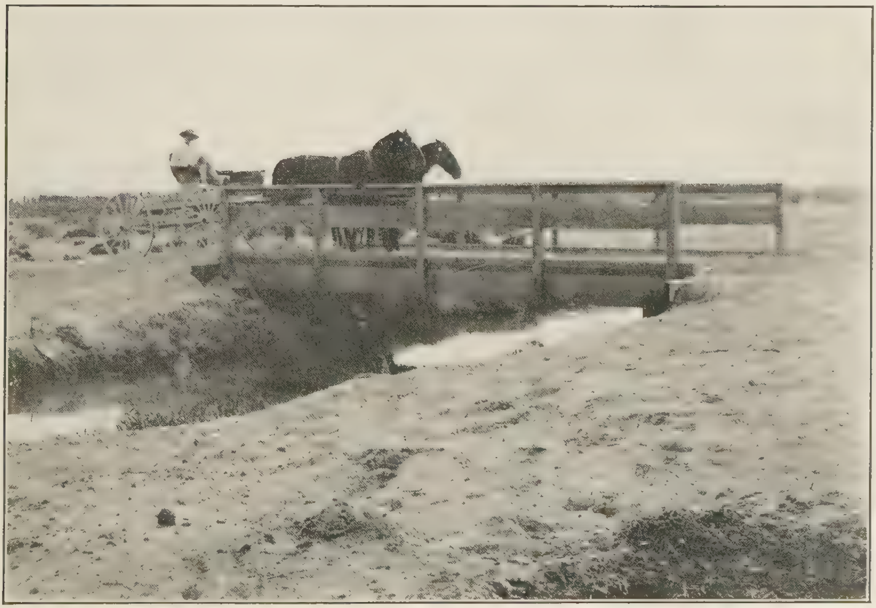


FIG. B.—Wooden stringer bridge. Truckee-Carson Project, Nev.

(*Facing page 346*)

is divided by intermediate piers into two or more spans (Figs. 141 and 142). The number of spans is usually determined to give the lowest total cost of bridge floor stringers and piers; for canals in soft earth, liable to be eroded by the irregular currents formed by piers, it may be desirable to use longer spans; but this is not usually justifiable if it materially increases the cost. The supports for the abutment ends of the bridge are either wooden or concrete abutment walls with side wings to retain the earth pressure and form the earth approach to the bridge floor (Fig. 142) or abutment sills located on the canal bank and supported on firm material or placed on posts or concrete footings (Fig. 141). A minimum difference in elevation between the lower edge of the stringer and the full water supply level in the canal of 6 inches and preferably 12 inches is necessary. The total length of the bridge floor, when supported on abutment sills, will be about equal to the top width of the canal; when supported on abutment walls the length is shorter; it is usually made about equal to the average width of the water cross-sectional area; this is usually desirable in order that there be no contraction in the waterway, which would produce a backing up of the water on the upstream side and an increase in velocity of flow under the bridge. Where this will not result in harmful erosion and when the loss of head is not detrimental, the distance between abutment walls may be made more nearly equal to the bottom width of the canal, and thus decrease the required length of bridge (Fig. 142). The selection between the abutment wall type and abutment sill type for the supports at the ends of wooden bridges is determined largely from the effect on the total cost of the bridge.

The width of the bridge will depend on its location and traffic condition. The width of farm bridges or bridges on roads subject to light traffic is usually 10 to 12 feet; of ordinary country road bridges, 14 to 16 feet; and of main county roads, 18 to 24 feet.

Examples of Multiple Span Wooden Stringer Highway Bridges.—Fig. 141 shows the standard type of multiple span highway bridge, designed for six bridges on the Dodson South Canal of the Milk River project, Montana. The total span is 62.5 feet for four of the bridges, 65 feet for one bridge, and 30 feet for the other. The detail dimensions are as follows:

Dimensions for Standard Multiple Span Highway Bridge on Dodson South Canal, Milk River Project, Montana

Total clear span in feet	Number and spacing of stringers		Number of pieces 3×12 inch by 16 feet flooring	Dimensions in feet		
	Number of stringers 12 feet long	Spacing inches		A	B	H
30	39	15⅜	32	10	10	8
62½	65	15⅜	64	10¼	10½	7⁴⁄₁₀
65	70	14⁷⁄₁₆	67	10½	11	6⁸⁄₁₀

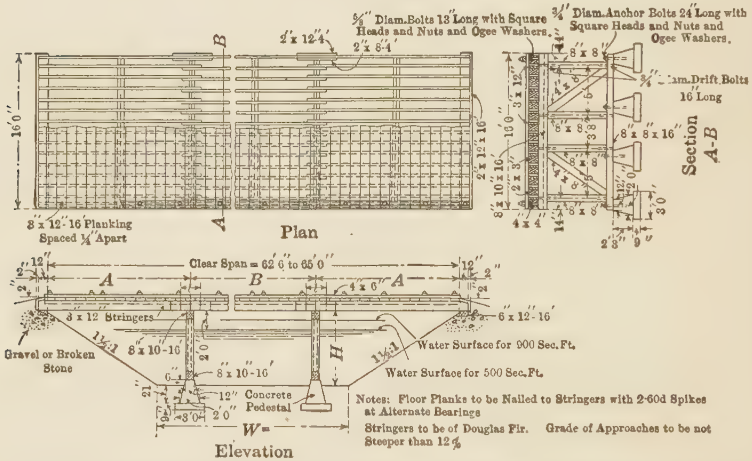


Fig. 141.—Multiple span wooden stringer highway bridge. Milk River Project, Mont.

Fig. 142 shows the standard multiple span highway bridge used by the Sacramento Valley Irrigation Co., California. Oregon pine is used for the stringers, flooring, sway bracing, bridging, fence guard and railing; and redwood for all other pieces. One line of bridging is used at the center of each span and spreader blocks placed between stringers at the ends on the middle bent cap. The following sizes of stringers are used: For 13- and 14-foot spans, 3 × 14-inch stringers; for 14-, 15-, 16- and 17-foot spans, 4 × 14-inch stringers; for 18-, 19- and 20-foot spans, 4 × 16-inch stringers.

Reinforced Concrete Bridges.—These have been used to a smaller extent than wooden bridges; the most common type is the simple flat slab, but for larger spans the T-beam slab, the

I-beam slab and the girder type have been used. To distribute the load and reduce impact on the bridge, the road-bed material, macadam, gravel or earth, is laid on the floor slab to a depth of at least 12 inches.

The reinforced concrete slab is usually built monolithically with the abutments. The abutments usually consist of the concrete abutment wall and wings. In some cases no wings are used and the slab is supported on abutment concrete sills. The total length of the bridge is determined from the considerations stated above for wooden stringer bridges. The division of the

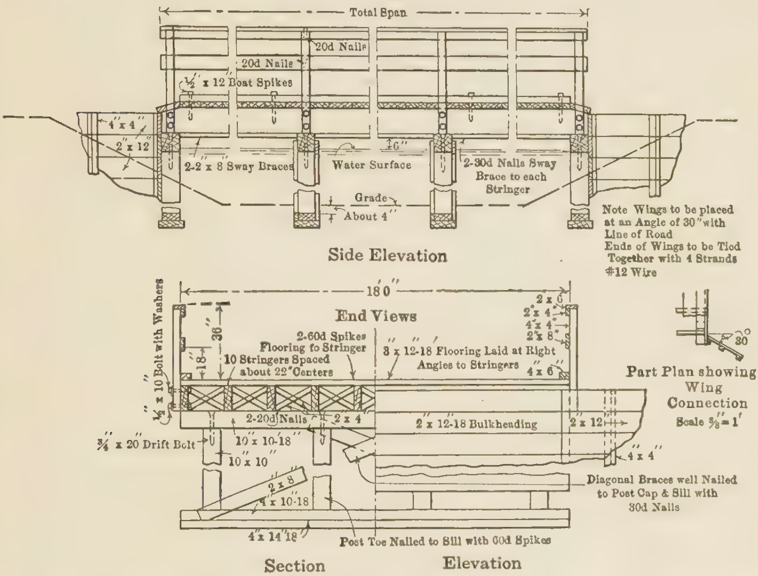


Fig. 142.—Multiple span wooden stringer highway bridge. Sacramento Valley Irrigation Co., Calif.

length into spans and the selection of the type of reinforced concrete bridge is an economic problem, which is worked out by cost comparisons. Within certain approximate limits the adaptability of the different types can be stated for general average conditions. For single span bridges of the usual width for country roads, 18 to 24 feet, the slab type is more economical than the other types, up to a maximum span of about 14 to 16 feet (Figs. 143 and 144). For larger single spans the T-beam or I-beam slab can be used economically up to about 24 to 30 feet. For larger spans and for narrow bridges whose width is less than

the span the reinforced girder type will be more economical. These limits indicate the selection of the type of bridge for a single span, but not the economic spans for multiple span bridge, which will depend on the height, design and cost of intermediate piers. This is illustrated by designs made for standard flat slab bridges of the Yolo Water and Power Co's. Irrigation System (Fig. 144); these designs show that for bridge lengths between abutment walls greater than 12 feet, ranging from 14 to 24 feet, it is more economical to use an intermediate pier. A comparison of the design of a two-span flat slab bridge, with that of a single span T-beam slab bridge, shows a small difference in

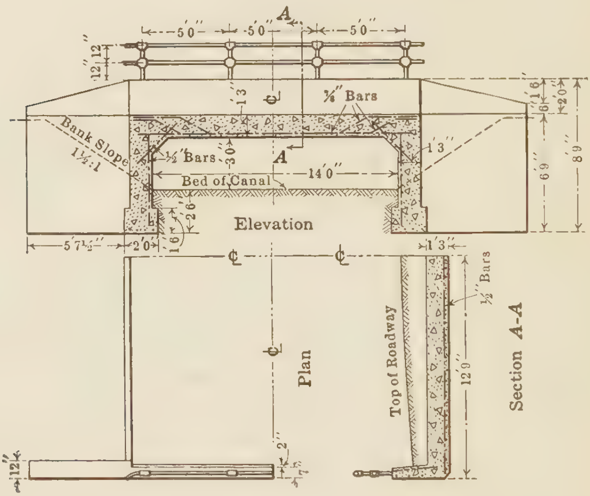
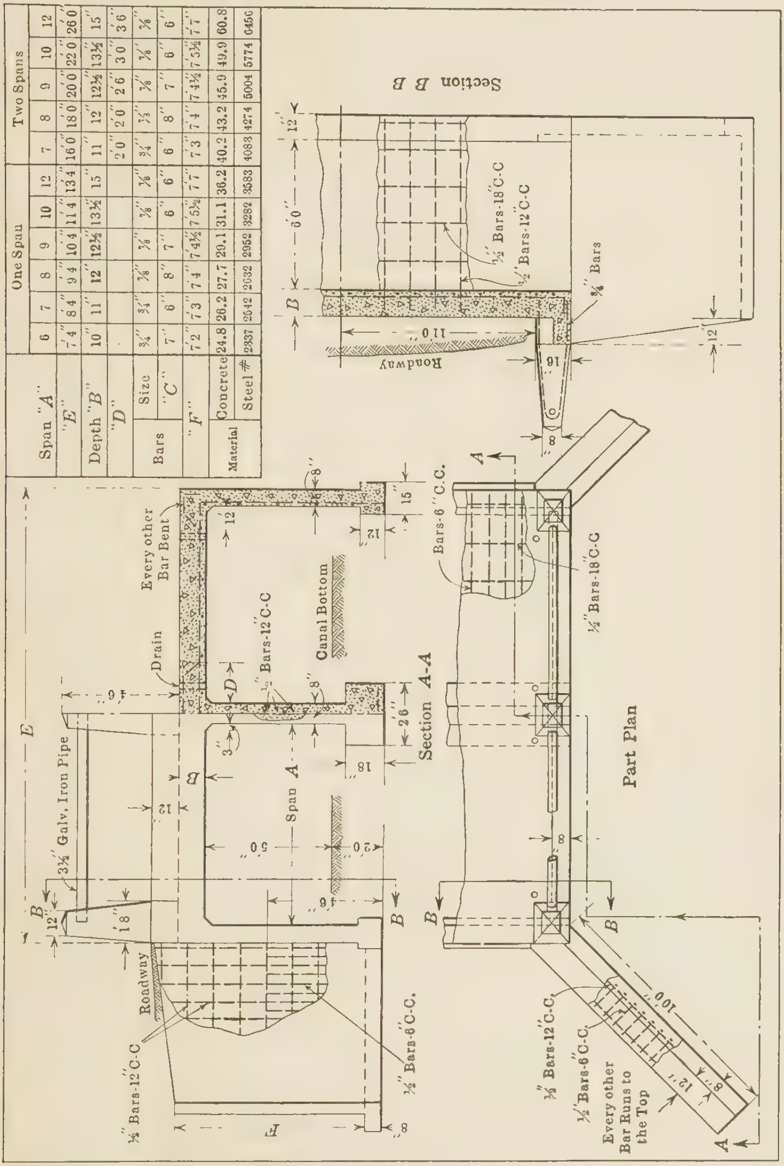


FIG. 143.—Flat slab bridge. Turlock Irrigation District, Calif.

cost in favor of the two-span design up to about 20 feet; but for a higher center pier of heavier design the cost would probably be in favor of the T-beam bridge.

Arch bridges of plain concrete and small spans, up to about 8 feet, are essentially culverts commonly formed of a full semi-circular arch, supported along each edge on an abutment or side wall. Their cost is usually greater than that of a box culvert, and they are seldom used on irrigation work. Arch bridges of reinforced concrete are adapted to wide spans, but the required head room for the rise of the arch is only obtainable where the canal is in comparatively deep cut; firm foundation for the

abutments is required. Their use is very limited on irrigation canals.



		One Span						Two Spans				
Span "A"		6	7	8	9	10	12	7	8	9	10	12
"E"		7'4"	8'4"	9'4"	10'4"	11'4"	13'4"	16'0"	18'0"	20'0"	22'0"	26'0"
Depth "B"		10"	11"	12	12½"	13½"	15"	11"	12"	12½"	13½"	15"
"D"		.						2'0"	2'0"	2'6"	3'0"	3'6"
Bars	Size	¾"	¾"	⅞"	⅞"	⅞"	⅞"	¾"	⅓"	⅞"	⅞"	⅞"
	"C"	7"	6"	8"	7"	6"	6"	6"	8"	7"	6"	6"
"F"		7'2"	7'3"	7'4"	7'4½"	7'5½"	7'7"	7'3"	7'4"	7'4½"	7'5½"	7'7"
Material	Concrete	24.8	26.2	27.7	29.1	31.1	36.2	40.2	43.2	45.9	49.9	60.8
	Steel #	2337	2542	2632	2952	3282	3583	4083	4274	5004	5774	6456

FIG. 144.—Standard county road bridge. Yolo Water & Power Co., Calif.

Truss bridges are occasionally used on wide canals in deep cut where they may be more economical than short span bridges, on

account of the height of intermediate piers. The bridge may be a combination wood and steel bridge, usually of the King post type for spans up to about 36 to 40 feet, and of the Howe truss type for greater spans, or may be a steel truss bridge, usually of the Warren or Pratt type. The pin-connected Pratt truss has the advantage of ease of erection.

DESIGN OF CULVERTS AND BRIDGES

Loads.—The use of culvert pipes, except those of reinforced concrete of large diameter, is usually based on experience and results obtained in practice. Large reinforced concrete pipes, box culverts and bridges must be designed to resist the stresses due to dead load, live load and impact.

The external pressure produced by these loads on the roof of a culvert is subject to uncertainties which do not exist usually with bridges. The resultant external pressure is dependent on a number of factors, of which the depth of backfill, its character and condition, the amount of moisture it contains, and the degree of compactness obtained in placing and tamping it are most important. The usual assumption is that the earth-fill distributes the load, so that the external pressure is exerted vertically and uniformly on the roof surface of a box culvert, or on the horizontal projection of a pipe culvert.

The dead load may include: (1) the weight of the bridge floor or culvert slabs; (2) the weight of the earth covering, at least in the case of culverts and concrete bridges; (3) the weight of the rails and track in case of a railroad crossing. The live load must be the maximum to which the structure will be subject. For country roads the maximum load will be a traction engine of 15, 20, or even 25 tons. For farm roads the heaviest load may be as much as 8 to 10 tons, in case of sugar beet hauling, but usually is under 6 tons. For bridges the live load is usually considered as an equivalent concentrated load. In Bulletin No. 45 of the Office of Public Roads, on Data for Use in Designing Culverts and Short Span Bridges, by Ch. H. Moorefield, the live load for slab top culverts and bridges up to a maximum span of 16 feet is assumed to be 2,000 pounds, concentrated at the center of the span, on a strip 1 foot wide. This is considered sufficient for all ordinary highway traffic up to and including a 15-ton

roller. To provide for impact, this live load is increased 50 per cent. For farm bridges, subject to the usual farm loads, the live load may be taken as half that given above, with an allowance for impact of 25 per cent.

For culverts with a depth of earth covering on the roof not less than half the span or width of slab and at least 3 feet, the live load may be considered as a uniformly distributed load of 150 pounds per square foot for heavy highway traffic, and 700 to 1,000 pounds for railroad traffic. To make an allowance for impact, the live load for a highway crossing may be increased 25 per cent. and for a railroad crossing 50 per cent.

The California State Highway Commission uses for standard slab culverts and for reinforced concrete bridges up to 20-foot span, the live load of a 20-ton engine on four wheels with 12-foot wheel base and wheels spaced 6 feet, center to center (rear tires 24 inches wide and front tires 12 inches wide). The rear weight is 14,000 pounds per wheel, and the front weight is 6,000 pounds per wheel.

Principles of Design of Culverts and Short Span Slab Bridges.—The following discussion is largely limited to the determination of stresses in pipe and box culverts and the design of short span flat slab bridges. For the structural design of the details and of abutments of piers, of reinforced concrete T-beam and I-beam slabs, of arches, wooden and steel truss bridges, standard textbooks may be consulted.

Pipe Culverts.—The results of the tests and investigations made by Prof. A. N. Talbot, described in Bulletin No. 22 of the University of Illinois, show that the external pressure on a properly bedded pipe may be considered as a vertical load distributed uniformly over the horizontal projection or outside diameter of the pipe. The lateral pressure may produce side restraint, which will decrease the bending moment on the pipe developed by the vertical load; but this effect is neglected and assumed to offset the increase in bending moment which may result from the unequal distribution of pressure obtained in practice. The vertical loading tends to shorten the vertical diameter of the pipe and lengthen the horizontal diameter, producing positive bending moments at the top and bottom of the pipe and equal negative bending moments at the two sides. Using the following notation for a pipe ring of unit length:

23

d = diameter of pipe.

W = total vertical load in pounds on the horizontal projection of the pipe ring.

M = bending moment for the pipe ring of unit length.

Then

$$M = 1/16Wd.$$

The reinforcement of reinforced concrete pipe, designed to resist the positive and negative bending moments must be shaped and placed so as to be near the inside face of the pipe at the top and bottom and near the outside at the two sides.

Rectangular Box Culverts.—Two methods of analysis are commonly used. The first method assumes a reaction for the bottom slab equal to the vertical load, and a lateral pressure on the sides equal to ⅓ of the vertical load; and the bending moments are obtained as for simply supported beams. The second is that presented by Turneaure and Maurer in their book on Principles of Reinforced Concrete, and is as follows for a uniform load (Fig. 145):

Let l_1 = width of culvert.

l_2 = height of culvert.

I_1 = moment of inertia of top and bottom assumed equal.

I_2 = moment of inertia of sides.

p = vertical load and foundation reaction per unit area.

$$M_a = \frac{pl_1^2}{8} \cdot \frac{\frac{1}{3}\frac{l_1}{I_1} + \frac{l_2}{I_2}}{\frac{l_1}{I_1} + \frac{l_2}{I_2}}$$

$$M_b = M_c = M_a - \frac{1}{8}pl_1^2$$

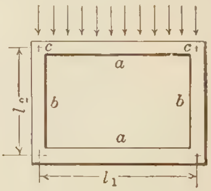


FIG. 145.

For a square culvert with uniform section:

$$M_a = \frac{1}{12}pl^2$$

$$M_b = -\frac{1}{24}pl^2$$

For equal vertical and lateral loads and a square culvert:

$$M_a = M_b = +\frac{1}{24}pl^2$$

$$M_c = -\frac{1}{12}pl^2$$

Reinforcement for negative moments at the corners is necessary. When it is omitted, as is often done at least for small

culverts up to about 4 feet span, then the bending moment for the four sides is obtained by considering them as simple beams or flat slabs.

Flat Slab for Short Span Bridges.—The following table is taken from Bulletin No. 45 of the Office of Public Roads, previously referred to. It gives the data necessary for designing flat slab bridges up to a span of 16 feet. The computations are made for the live load of 2,000 pounds, concentrated at the center on a strip 1 foot wide, plus 50 per cent. to allow for impact and a dead load, uniformly distributed, of 150 pounds per square foot, plus the weight of the slab. The minimum allowable cushion of roadway material on the top of the slab is 12 inches. The safe strength of the concrete in compression is 700 pounds per square inch; of the steel in tension, 16,000 pounds per square inch. The amount of reinforcement is 0.75 of 1 per cent.

DATA FOR DESIGNING FLAT SLAB BRIDGE

(From Bull. 45, Office Public Roads, U.S.D.A.)

Span, feet	Depth of slab to C.G. of steel, inches	Total depth of slab, inches	Area of steel per foot of width, sq. in.	Suggestions as to size and spacing of reinforcing bars
2	4½	6	0.360	½ inch square, 8 inches center to center, or ½ inch round, 6 inches center to center.
3	5½	7	0.450	½ inch square, 6 inches center to center, or ½ inch round, 5 inches center to center.
4	6	7½	0.573	½ inch square, 5½ inches center to center, or ⅝ inch round, 7 inches center to center.
5	6½	8	0.585	½ inch square, 5 inches center to center, or ⅝ inch round, 6 inches center to center.
6	7½	9	0.653	⅝ inch square, 7 inches center to center, or ¾ inch round, 8 inches center to center.
7	8	9½	0.720	⅝ inch square, 6½ inches center to center, or ¾ inch round, 7 inches center to center.
8	9	10½	0.810	⅝ inch square, 5¾ inches center to center, or ¾ inch round, 6½ inches center to center.
9	10	11½	0.900	⅝ inch square, 5¼ inches center to center, or ¾ inch round, 5¾ inches center to center.
10	10½	12	0.945	⅝ inch square, 5 inches center to center, or ¾ inch round, 5½ inches center to center.
11	11	12½	0.990	¾ inch square, 6¾ inches center to center, or ⅞ inch round, 7¼ inches center to center.
12	11½	13½	1.062	¾ inch square, 6¼ inches center to center, or ⅞ inch round, 6¾ inches center to center.
13	12½	14½	1.134	¾ inch square, 6 inches center to center, or ⅞ inch round, 6¼ inches center to center.
14	13½	15½	1.206	¾ inch square, 5½ inches center to center, or ⅞ inch round, 6 inches center to center.
15	14	16	1.269	¾ inch square, 5¼ inches center to center, or ⅞ round, 5¾ inches center to center.
16	15	17	1.350	¾ inch square, 5 inches center to center, or ⅞ inch round, 5¼ inches center to center.

CHAPTER XII

SPECIAL TYPES OF DISTRIBUTION SYSTEMS: WOODEN FLUME, WOODEN PIPE, AND CEMENT PIPE DISTRIBUTION SYSTEMS

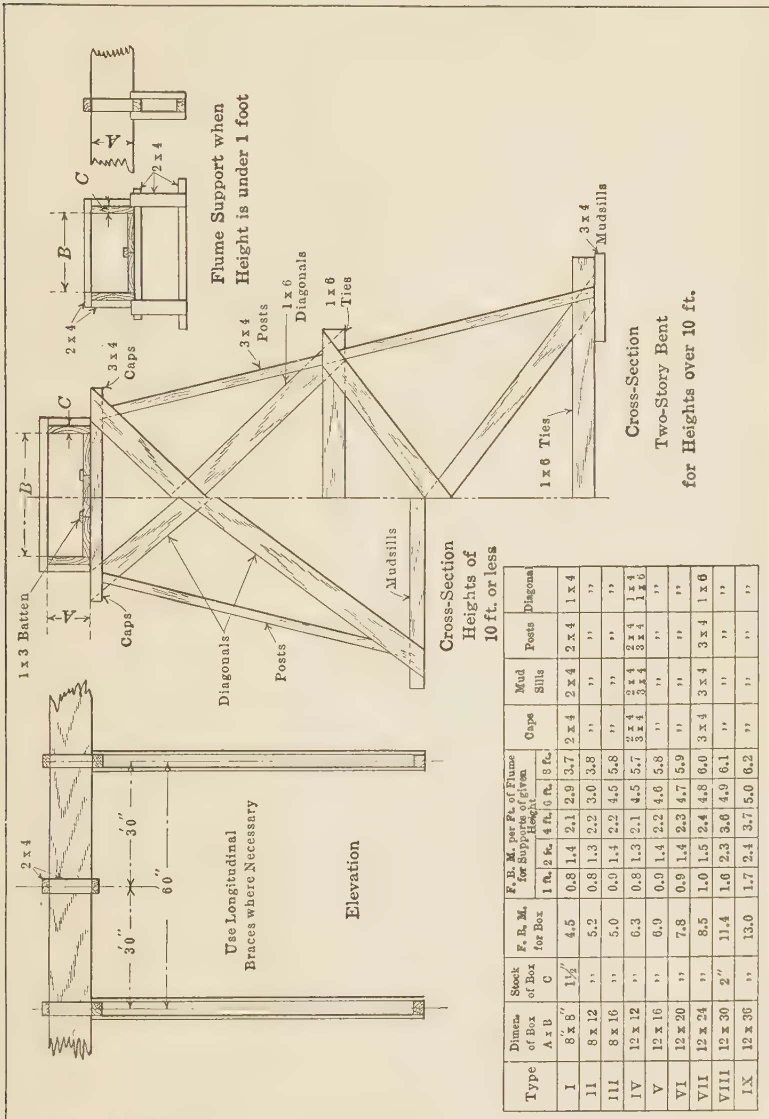
Special Conditions Favorable for these Systems.—The great majority of irrigation distribution systems consist of open canals or ditches in earth, but there are special conditions and difficulties which have led to the construction of a number of smaller irrigation systems usually for areas under 10,000 acres in extent, built largely or entirely either of wooden flumes, or wooden pipes as in some sections of eastern Washington, southern Idaho, northeast Oregon, and British Columbia; or of cement pipe as used for many years in southern California and more recently in the above named states. Some of these special conditions are the following:

1. The topography of the country, although not especially rough, is irregular, having no general gradual slope and no well-formed continuous ridges. This requires that laterals pass successively from ridges to depressions, making it necessary to use either fluming or pipes for at least a considerable portion of the laterals.

2. The topography is rough and steep, which for an open ditch system would require laterals placed along the irregular contours with other laterals running down the steep grades along broken ridges, with numerous drops or chutes to absorb the excess fall or concrete lined canals to resist the high velocities, and numerous siphons or elevated flumes to cross the depressions. These conditions may be specially favorable to pipe or flume systems, as the steep grades would permit the use of small size pipes or flumes, which may be more economical than open canal laterals. Pipes are usually preferable to flumes, especially for deep depressions which would require flumes on high trestles.

3. The volumes of water to be conveyed are small, the water is valuable, and the seepage losses must be prevented. The choice is then between cement lined canal, flumes, or pipes. These conditions are obtained when the water is valuable, due to its

scarcity or to the high cost of development as by pumping, storage, or difficult and expensive construction.



Type	Dimen. of Box A x B	Stock of Box C	F. B. M. for Box	F. B. M. per Ft. of Flume for Supports of given Height					Caps	Mud Sills	Posts	Diagonal
				1 ft.	2 ft.	4 ft.	6 ft.	8 ft.				
I	8″ x 8″	1½″	4.5	0.8	1.4	2.1	2.9	3.7	2 x 4	2 x 4	2 x 4	1 x 4
II	8 x 12	,,	5.2	0.8	1.3	2.2	3.0	3.8	,,	,,	,,	,,
III	8 x 16	,,	5.0	0.9	1.4	2.2	4.5	5.8	,,	,,	,,	,,
IV	12 x 12	,,	6.3	0.8	1.3	2.1	4.5	5.7	2 x 4 3 x 4	2 x 4 3 x 4	2 x 4 3 x 4	1 x 4 1 x 6
V	12 x 16	,,	6.9	0.9	1.4	2.2	4.6	5.8	,,	,,	,,	,,
VI	12 x 20	,,	7.8	0.9	1.4	2.3	4.7	5.9	,,	,,	,,	,,
VII	12 x 24	,,	8.5	1.0	1.5	2.4	4.8	6.0	3 x 4	3 x 4	3 x 4	1 x 6
VIII	12 x 30	2″	11.4	1.6	2.3	3.6	4.9	6.1	,,	,,	,,	,,
IX	12 x 36	,,	13.0	1.7	2.4	3.7	5.0	6.2	,,	,,	,,	,,

FIG. 146.—Standard small wooden flume. Tieton Project, Wash.

In addition to their adaptability to the above conditions, pipe systems have special advantages; they do away with bridges and

other structures required on open canals; they do not occupy any land which is wasted; they do not collect dirt or rubbish that collects in open canals. On account of the cost, pipe systems are, however, feasible only for the conditions stated above.

Wooden Flume Systems.—A complete wooden flume system will consist of main flumes and lateral flumes, supported on mudsills or elevated on trestles and ranging from the larger sizes down to the smaller sizes, 8×8 inches in cross section. The design of flumes has been fully discussed in Vol. II, Chapter IX, in which a number of examples are given. Examples of small flumes are also shown in Vol. I, Chapter VII, on Farm Ditches and Structures. Standard sizes of small wooden flumes, used on the Tieton project in eastern Washington, are given in the accompanying drawing (Fig. 146). On this project the rolling topography of a large part of the land has required the extensive use of concrete pipes and wooden flumes for sub-laterals.

Flume systems are expensive, both in first cost and ultimate costs, especially when the average height of trestles is large, and may be less economical than wooden pipes.

Pipe Systems.—The kinds of pipe most commonly used are wood-banded pipe and cement-mortar or cement-concrete, hand-tamped pipe. Vitrified clay pipe has been used only to a very limited extent for a few pipe lines on some projects in southern California; it is usually more expensive than the hand-tamped cement and is more difficult to lay. Steel pipe has been used for distribution lines on a few systems of the high-pressure type described further. Reinforced concrete pipe has been used on a number of projects for separate pipe lines and in a very few cases for the larger part of the distribution system; they are well adapted to topographic conditions which produce depressions deeper than can be crossed with the non-reinforced pipe, and for maximum pressure heads not exceeding about 100 feet.

The cement hand-tamped pipe, extensively used in southern California and in more recent years used for the entire distribution system of at least one project in British Columbia, one in Colorado, one in Idaho, and for a large part of a number of smaller systems in other states, is the cement-mortar or cement-concrete pipe, hand-tamped in metal moulds, made in 2-foot lengths, which are joined in the trench. The sizes commonly used are 6-, 8- and 10-inch, inside diameter, for private distributing lines, and 10 to 30 or even 36 and 48 inches for the laterals and main lines of the

irrigation system. The properties and method of manufacture of this kind of pipe have been fully described in Vol. II, Chapter X. The use of the pipe is limited to low pressures for which safe values have been given; it is therefore used only for the low-pressure type of system described below.

The wooden pipe which is used extensively on a number of systems is best adapted to pressures ranging from low pressures, greater, however, than those for which cement pipe can be used, up to high pressures of usually not over 200 to 300 feet. The pipe is machine-banded or wire-wound pipe for sizes up to about 24 inches in diameter, above which continuous wood stave pipe is used. The use, properties and life of wood pipe have been fully presented in Vol. II, Chapter X.

Types of Pipe Distribution Systems.—Pipe systems may be classified as high-pressure systems and low-pressure systems, depending on the kind of pipe and on the topography. There may be no sharp distinguishing factors to determine which type should be used, and one type may merge into the other. In general the high-pressure type is made of wood pipe, in some cases steel pipe, and is similar to the distribution system of a domestic water supply system; the low-pressure type is generally made of cement mortar or cement concrete pipe, in some cases combined with wood pipe or reinforced concrete pipe, and in its operation is in some respects similar to a gravity open ditch system.

Relative Cost of Wooden Flume and Pipe Distribution Systems.—From an extensive examination and study of a large number of wooden flume systems and wooden pipe systems mostly in British Columbia, of a complete concrete pipe system in British Columbia, and others in southern California, the following approximate total cost of construction, including engineering and administration expenses, but not interest during period of construction, has been obtained.

A complete wooden flume distribution system, delivering water to 5- or 10-acre tracts, will cost \$20 to \$30 per acre, where the maximum depth of depressions to be crossed will not exceed about 20 feet for generally favorable conditions, and \$30 to \$40 for depressions up to 30 to 50 feet maximum depth and rough topography.

A low-pressure distribution system made entirely of hand-tamped cement pipe laterals and distributaries, delivering water to each 5- or 10-acre tract, will cost from \$30 to \$40 an acre.

A high-pressure distribution system made entirely of wood pipes, delivering water to each 5- or 10-acre tract, will cost from about $40 to $75 an acre. The larger cost was obtained in British Columbia for projects irrigating steep rolling lands where several pipe lines were under pressure heads exceeding 200 feet, and where the prices of wood-banded pipe were about 50 per cent. greater than in California, Oregon, Washington and Idaho.

These average costs are necessarily only roughly approximate. They do not include the cost of the main diversion canal, flume or pipe from the point of diversion to the beginning of the distribution system, which will vary greatly, depending on the length, topographic condition, difficulty of construction and type of construction.

A comparison of cost must not only consider first cost, but also ultimate cost, which is dependent on the durability or life of the system and includes depreciation, annual maintenance and repairs, and the interest on the capital invested.

The average useful life of a well constructed wooden flume system ranges from about 10 to 12 years when built of fir and 13 to 15 years when built of redwood; during this time renewal of mud sills, repairs, tarring and calking will represent the items of maintenance.

The life of a machine-banded wooden pipe, when full only part of the time and buried in the soil, is very variable; it depends largely on the pressure, the kind of wood, the coating, and on the character of the soil. In some instances, notably when placed in dry porous soils, pipes have had to be renewed in 4 or 5 years or even less. Pipes kept full only part of the time and made of selected fir, free from sapwood, well coated on the outside, will give a probable average useful life for the system of 10 to 15 years, and when made of redwood 15 to 20 years. Pipe lines which can be kept constantly full, buried sufficiently deep to prevent freezing, and under sufficient head to obtain saturation of the wood, preferably not less than 30 or 40 feet head, will have a much longer life. Where these conditions can be approximated for most of the system, the average life of the system will probably be twice the range of values given above.

The items to be considered in the study of the ultimate cost for flume and pipe systems may be summarized by the following approximate figures:

For wooden flumes, useful life 10 to 12 years:

	Per cent.
Average annual maintenance: repairs, tarring, calking, etc.	5
Sinking fund for renewals	9
Interest on capital invested	6
Total annual cost	20

For wooden pipes empty part of the time, useful life 10 to 15 years for fir, 15 to 20 years for redwood:

	Per cent. for fir	Per cent. for redwood
Maintenance and repairs	2	2
Sinking fund for renewal	7	5
Interest on capital invested	6	6
Total annual cost	15	13

For wooden pipes always full, useful life 20 to 30 years for fir, 30 to 40 years for redwood:

	Per cent. for fir	Per cent. for redwood
Maintenance and repairs	1	1
Sinking fund for renewal	4	3
Interest on capital	6	6
Total annual cost	11	10

These figures are in the approximate ratio of 1, 1.4 and 2; they indicate that a wooden pipe system, kept full of water only part of the time will be more economical than a flume system, as long as its first cost is less than 1.4 times the cost of the flume system, and that a wooden pipe system, in which the pipe lines are always full, will be more economical when the cost does not exceed twice the cost of a flume system.

High-pressure Pipe System.—High-pressure systems are used when the land is steep and irregular, producing high pressures on the pipes. One form of such a system consists either of a high line main canal or main pipe line located along the upper part of the land to be irrigated, and from which the pipe laterals take out to supply sub-laterals, or to directly supply the farms or orchards in the valley below. Another form consists of a main pressure pipe line placed in the trough of the valley, with branches extending up the sides of the valley, along the ridges, where possible, to supply smaller laterals, or the farms or orchards. The water is delivered from the supply lateral to the irrigator either through a valve takeout, located at the high point of the land served, or

when the consumer has a pressure pipe system of distribution, through a service connection made between the supply lateral and the main pipe of the irrigator's system (Fig. 147). This direct connection gives the consumer water under pressure, but requires a domestic supply meter for the measurement of the water. Where the delivery is made through a valve takeout, a measuring weir box can be formed around the valve, as illustrated by the pressure takeout box for the low-pressure system (Fig. 152).

When the main pipe line is placed along the trough of the valley, with laterals and sub-laterals extending laterally to supply the farm units on each side, the system is similar to a domestic supply system. A few such systems in the northwest supply irrigation and domestic water through the same pipes. Such a combination

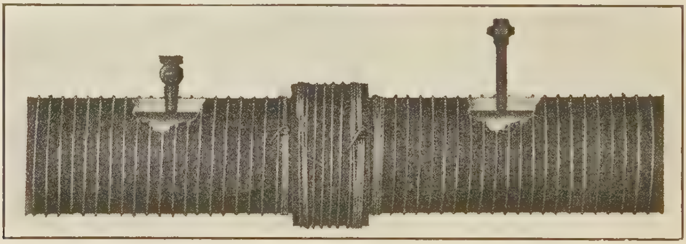


Fig. 147.—Wire wound pipe with two types of service connections. Nipple with Union on the right; Standard Corporation cock on the left. (Pacific Coast Pipe Co.)

maintains the pipe lines constantly full throughout the year, which is favorable for a long life, but it requires that the pipe lines be protected against freezing by being buried to a depth of usually not over 3 feet, and where it is necessary to carry the pipe above the ground it must be protected by means of boxing surrounding the pipe with the space in between filled with well packed sawdust. It is not always possible and often not desirable to carry the domestic water and the irrigation water in the same pipes. In many cases the source of supply may be so polluted that either another source is desirable for the domestic water, or the water must be treated or filtered to purify it, in which case it would be uneconomical and not practicable to purify the irrigation water as well as the domestic water.

High-pressure systems of the above description have been constructed for some of the orchard lands of Washington, Idaho,

British Columbia and Southern California. The design of the system is similar to that of a domestic supply distribution system, for which text-books on Public Water Supplies and on Hydraulics may be consulted.

LOW-PRESSURE SYSTEM

Conditions Favorable for Its Use.—This type is well adapted to land with a good slope, or to rolling land with few or no deep depressions to cause pressures greater than the ordinary cement pipe will stand. An average slope of at least 10 to 20 feet per mile is desirable to avoid excessively large pipes, although less may be used. The depressions should in general produce pressure heads not greater than 10 to 15 feet; there may be some deeper depressions for which special pipe may be used. The system can be used advantageously on land with steep grades, where it is feasible to so locate the pipe lines and regulate the flow in them, that the pipes are not depressed too far below the hydraulic grade line; this is usually obtainable for regular side-hill land, not broken up, and with a fairly uniform slope.

Parts of Low-pressure System and Principles of Design.—Such a system will have a main canal, usually concrete lined, or a main supply pipe placed along either the high boundary or the main ridge of the tract to be irrigated, and of smaller pipe-line laterals or branches heading at the main canal or pipe from which they divert the water to deliver it to the farmers or to smaller sub-laterals. All the pipe laterals are open at their lower end; they are generally made of uniform full capacity for their entire length and are extended to discharge into a waste or drainage channel. The open end, the uniform capacity and the open overflow regulating boxes along the pipe line insure that the pressures cannot be increased beyond the pressures for which the pipe line was designed. The flow in the pipe lines approaches a gravity flow similar to some extent with that of an open ditch. To meet this requirement, advantage must be taken of the topography by locating the pipe lines to conform in general with the location of an open-ditch system. When the land forms ridges, the main pipe laterals are located on the main ridges and the sub-laterals are located along the smaller ridges. When depressions have to be crossed which produces pressures too great for cement pipe, reinforced concrete, wood or steel pipe may be used. When the slope of the country gives to a pipe line a uniform grade from the

upper end to the outlet, the pipe line is essentially a chute, from which the water is usually taken out by means of overflow take-out boxes, located at points of delivery or points of connection with sub-laterals. These boxes fulfill the same purposes as check gates on open canals. They check the flow and raise the water level up to the required height for diversion or delivery. A single box of this type may, on flat grades, regulate the pressure for deliveries at two or more points above, in which case it may be called a pressure regulating box. When there is distinct change in grade from a steep to a flat grade, or when depressions

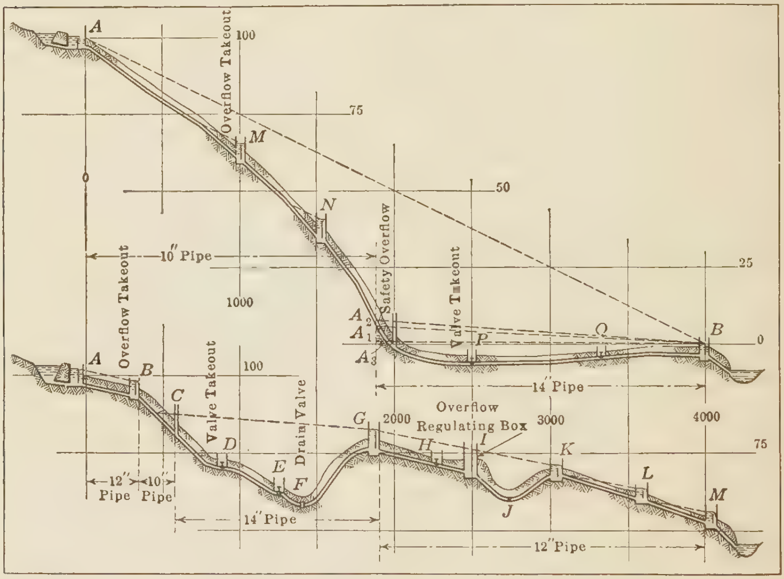


FIG. 148.—Profiles of pipe lines for low pressure pipe systems.

occur in the length of the pipe line, the size of the pipes, the position of overflow takeout boxes and pressure regulating boxes must be determined only after a careful consideration of the position of the hydraulic grade lines, not only for full supply but also for partial supply. Deliveries from the sections of pipe line, which are across depressions and therefore under pressure, are made through a valve takeout box.

Some of the features of design are illustrated by the accompanying diagram (Fig. 148) containing two sketch profiles, made up to present the range of problems which may occur in practice.

The method of design and computations as used for similar problems on the project of the Fruitlands Kamloops Irrigation project in British Columbia are as follows:

The first profile is for a pipe line, so situated that the upper part irrigates foothill land on a steep slope and the lower part irrigates flat bench land. Assume that the pipe line is 4,000 feet long, and serves a strip of land extending on each side about ¼ of a mile, including a total acreage of about 240 acres. The full supply capacity of the pipe is based on a maximum use of 1 second-foot to 80 acres, and is therefore 3 second-feet. If the pipe line is dimensioned to carry this flow, for the hydraulic grade line connecting the upper end of the pipe A with the lower end B, the grade is 25 feet in 1,000 feet, and the diameter is 10 inches (see Vol. I, page 157). This is the minimum diameter that could be used, but for the topographic conditions existing in this example the maximum pressure head would be about 50 feet, which is greater than the pipe could stand. Another important consideration is the effect of variations of flow in the pipe line on the means and regulation of deliveries from the pipe. For condition of full flow the entire pipe line would be under pressure, and delivery at any point on the pipe would be made through a valve or pressure takeout. But if the pipe line is operated at partial capacity, such as, for instance, ⅓ capacity, the corresponding hydraulic gradient is 3 feet per 1,000 feet and the hydraulic grade line occupies then the position A_1–B; this results in a gravity flow in the section of pipe line A–A_1 and pressure flow in A_1–B. For this condition of partial flow any takeout above A_1, which for full flow was under pressure, would now require some means of checking the flow in the pipe to make a delivery. While it would be possible to check the flow in this section of pipe by the use of gate valves, the cost would usually be excessive and in the majority of cases would create pressures in the pipe lines much greater than the pipe could stand.

The above considerations therefore show that the dimensioning of the pipe lines must be based on two requirements:

First.—Any variation in flow should produce a relatively small change in the position of the hydraulic grade line.

Second.—A flat hydraulic grade line should be used where by doing so the pressure head on the pipe may be decreased to what the pipe will safely stand. For instance, if in the above example a 14-inch pipe is used, the corresponding

hydraulic gradient for the full flow of 3 second-feet is about 3.7 feet per 1,000 feet (A_2–B), and for partial flow of 1 second-foot is about 0.5 foot per 1,000 feet (A_3–B). This solution gives a maximum pressure head of only 12 feet, and only a short section of pipe A_2–A_3, in which there may be either gravity flow or pressure flow, depending on the volume of flow in the pipe line. The flow in the pipe section above A_2, from A to A_2, will then always be gravity flow and takeouts from it at M and N will be of the gravity overpour type and the pipe line sub-sections A to M, M to N, and N to A_2 can then be dimensioned of smaller diameter for the corresponding hydraulic gradients. For these sections the hydraulic gradients, measured from A to M, M to N, and N to A_2, are greater than required for a 10-inch pipe, but not great enough for an 8-inch pipe, which is the next smallest size; therefore, 10-inch pipe will be used and the hydraulic gradients corresponding to full flow occupy then the lower positions shown in the profile. The flow in the pipe sections below A_3, from A_3 to B, will then always be pressure flow, and takeouts from it at P and Q will be of the pressure valve takeout type. It is possible to admit into the pipe line a greater flow than that for which it is designed; but this would result in a rise of the hydraulic grade line (A_2–B), with perhaps a corresponding dangerous increase in pressure head. To avoid this a safety overflow stand should be placed near A_2, with the overflow crest at an elevation a little higher than A_2.

The second sketch profile is worked out in the same manner by a consideration of the above requirements, such that the sections of pipe line in which there will be a variation from gravity to pressure flow and *vice versa* will be confined to short sections, which include no points of delivery, or in which the variations in hydraulic grade line are so small that an overflow pressure regulating box of not excessive height, such as at I, can be used to check the flow for the valve takeout H above it, when the pipe line is operated at partial flow.

This type of system requires relatively large pipes, when compared with a pressure pipe system, but it is made of hand-tamped cement-mortar pipe whose relative cost is much smaller than that of other kinds of pipes required for pressure pipe systems. Under topographic conditions favorable to its installation, it will be more economical and has the advantage of automatic regulation. This advantage may be further explained by a considera-

tion of the second profile. This pipe line is designed of uniform capacity from the upper end to the lower end. Delivery of water at overflow takeout boxes is practically self-regulating, because of the overflow provision, and delivery through valve pressure takeout boxes is also practically self-regulating, because the small variation in hydraulic grade lines insures nearly constant pressure heads.

Accessories to Low-pressure Systems.—These will include: Takeouts at the head of all pipe lines, either from the main canal or from the main supply pipe; pressure overflow regulating boxes; division boxes at points of connection with laterals; delivery boxes; measuring boxes; air stand pipes and blow-offs.

The takeout from a main concrete canal will consist in the simplest case of a connection through one bank of the canal in which the inlet of the pipe may be connected directly to the sloping side wall of the canal or may be formed with inlet wings, straight or warped. The floor may have to be depressed to bring the inlet well below the lowest operating water level in the canal which may be regulated with a check gate. The flow through the inlet is regulated by a gate in front of which a screen may be necessary to prevent the entrance of floating material. It will usually be desirable to combine the takeout with a measuring box, as shown by the takeout measuring box used on the Kamloops Fruitlands system (Fig. 149). The takeout from a main concrete pipe is a large division box of the type described below.

A pressure regulating box is formed of a rectangular box, divided into two compartments by an overflow wall. The floor of the box is placed at about the same level or a little lower than the bottom of the pipe line, which enters the upstream compartment through an opening in one end wall of the box and takes out through an opening in the other end wall. At the lower part of the overflow division wall is an opening of about the same diameter as that of the pipe, regulated by a gate. Where there is ample grade a smaller sized opening may be used to decrease the cost of the gate. When the gate is closed the water rises in the upstream part of the box and pours over the overflow wall back into the pipe. The elevation of the crest of the overpour wall is determined from the height to which it is desired to raised the water level. The height from the crest of the overpour wall to the top of the sides of the box must be sufficient to give the depth of overpour required to carry the entire flow of the pipe. The

structure is most commonly used as an overflow delivery and measuring box, described further (Figs. 153 and 154).

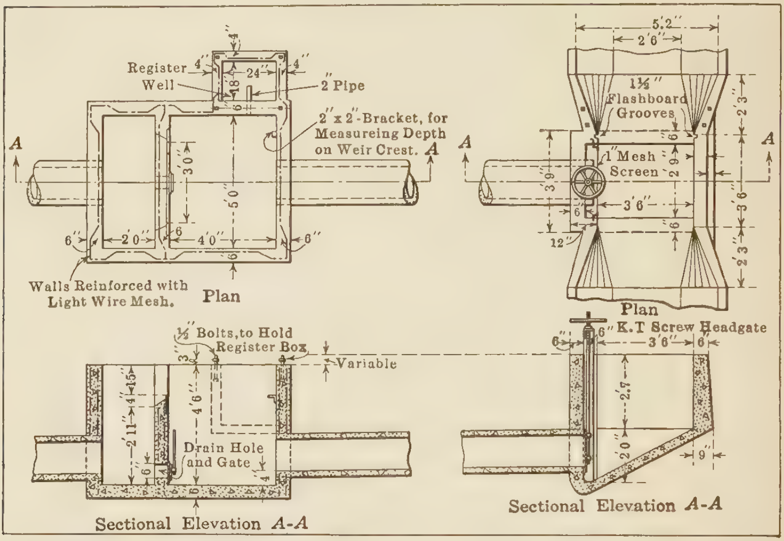


FIG. 149.—Takeout or lateral headgate from concrete lined canal. Kamloops Fruitlands Irrigation system, British Columbia.

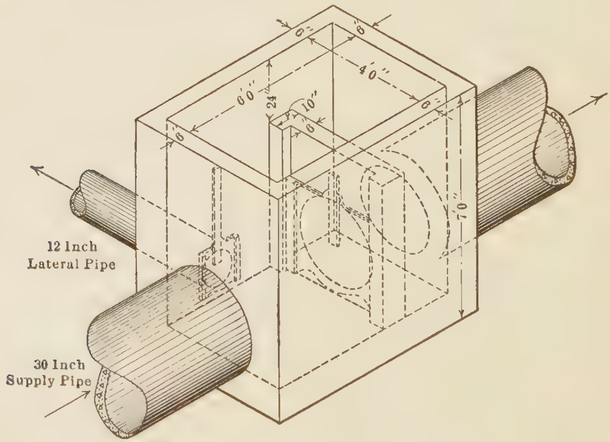


FIG. 150.—Pipe turnout. Covina Irrigation Co., Calif.

A division box is usually formed as a regulating pressure box with the branch connection made in the upstream compartment of the box. The connection may be made directly with the head of

the branch pipe line placed in the side wall, as illustrated by the pipe turnout of the Covina Irrigation Co. of Southern California (Fig. 150). It will usually be preferable to introduce provision for measuring the water by inserting a weir or orifice in the side wall for each branch with a receiving basin or compartment, to which the branch will be connected. Weirs, without

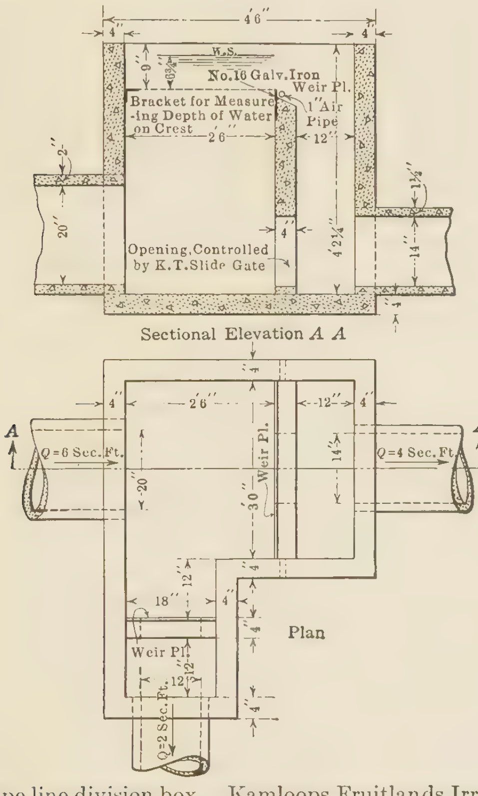


FIG. 151.—Pipe line division box. Kamloops Fruitlands Irrigation System British Columbia.

end contractions, with lengths proportionate to the discharges in the branches, are usually preferable, for they will maintain automatically the proper division of the flow, irrespective of variations in the flow. This form of structure is illustrated by the type of division box used by the Kamloops Fruitlands system (Fig. 151).

The delivery box through which the irrigator is served will preferably be formed to be used also as a measuring box. Two types

of boxes are used, depending on whether the delivery is made from a pipe line or section of pipe line in which there is a free gravity flow or from one which is under pressure. The first type is *the overflow delivery measuring box* and the second type is *the valve or pressure delivery measuring box.* The first type is an overflow pressure regulating box with an orifice delivery through a side

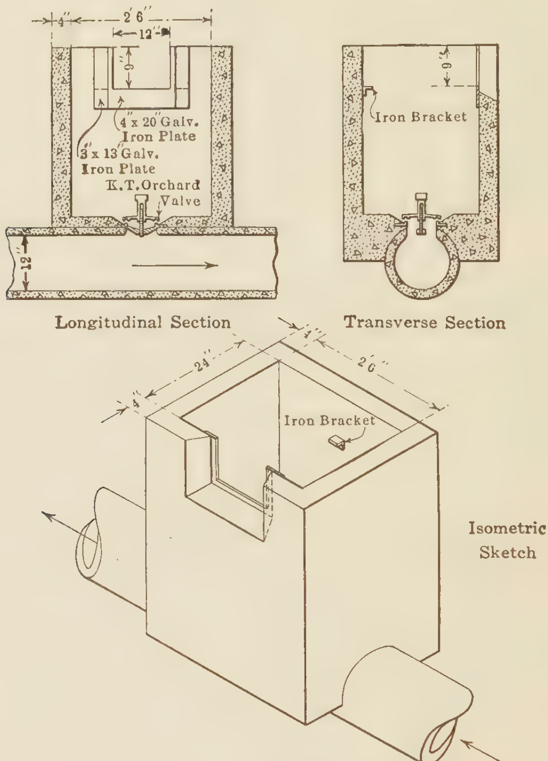


Fig. 152.—Typical weir box for valve takeout from pressure pipe line. Kamloops Fruitlands Irrigation System, British Columbia.

wall of the upstream compartment. A good example is the box used on the Kamloops Fruitlands system—also that used by the Azusa Irrigation Co. in Southern California (Fig. 154). The second type is made by cutting a hole in the top of the supply pipe, on top of which is cemented a valve, of the type used for orchard or farm cement pipe distribution systems, described in Vol. I, Chapter VII; around it is placed or built a measuring weir

box. This type of valve delivery box is illustrated by that used by the Kamloops Fruitlands system (Fig. 152). *Air stands* are necessary at all summits and at convex beds when the pipe approaches the hydraulic grade line. At the summits there are usually pressure regulating delivery boxes or division boxes. It is desirable to place an air stand within a short distance downstream of each of these boxes to give an escape to the air carried in by the water pouring over the overflow wall. At these points and at convex bends air stands are commonly built of two or more sections of cement pipes, placed vertically with the lower end cemented around a hole cut in the pipe line; these stands are in some cases as much as 12 to 16 feet in height. Failure to provide air stands of ample cross section will result in water-hammer, which will split the pipes. *Blow-offs* or drain valves at the lowest points are necessary to empty the pipe and flush out deposited material. It is especially important that they be provided in cold climates when the pipe lines are not buried below the depth of ground freezing. They are usually made of orchard or alfalfa valves, described in Vol. I, Chapter VII, cemented to the pipe at a hole cut near the bottom of the pipe.

Delivery and Miner's Inch Overflow Box Used by the Kamloops Fruitlands Co., in British Columbia (Fig. 153).—This box is similar to the boxes used on several systems in southern California, of which that used by the Azusa Irrigation Co., described below, is an example. The Kamloops Fruitlands box is designed to deliver up to 1 cubic foot per second, through a miner's inch orifice plate, placed in one of the side walls of the upstream compartment of the box, with the center of the orifice 8 inches below the crest of the overflow wall, which is the pressure head required for the British Columbia miner's inch. The size of the orifice is made adjustable by a slide working in grooves formed by two cover plates. The height of the opening is $4\frac{1}{2}$ inches and a width of 8 inches is required to give 36 miner's inches, which is approximately the equivalent of a cubic foot per second.

The water level in the upstream compartment is made to rise level with the overflow crest, by adjusting the gate in the overflow wall; the pressure on the orifice is regulated and maintained more or less constant by the overflow wall. A small excess is usually allowed to spill over the overflow wall. An increase in flow in the pipe line will increase the depth of overpour water and increase the volume delivered, but by proper operation the water

level may be kept fairly constant. The accuracy of the measurement will depend on the quantity spilling over the crest and the length of the overflow crest. A moderate increase in pressure will not affect the accuracy very greatly, especially as in this case an 8-inch pressure head is used; for instance, an increase in pressure head of 1 inch or 12 per cent. will increase the volume delivered 6 per cent. It would be feasible to use a weir plate in the place of the miner's inch plate, but with a weir plate the quantity of water delivered could not be adjusted as easily, and

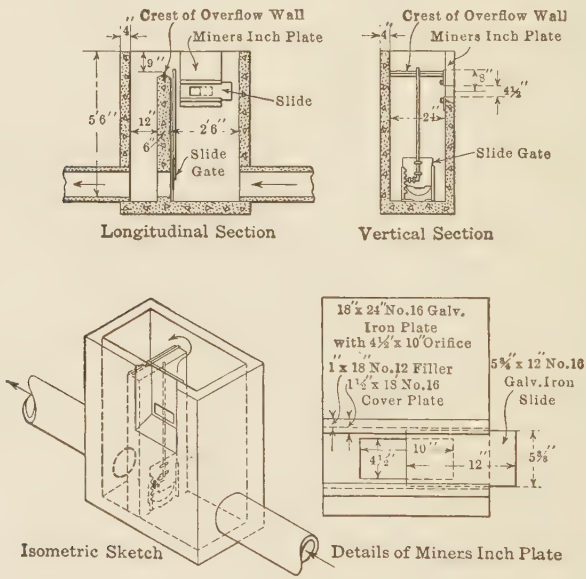


FIG. 153.—Delivery and miner's inch overflow box for takeout from pipe line under no pressure. Kamloops Fruitlands Irrigation System, British Columbia.

the increase in flow would be affected to a much greater extent by an increase in depth of water on the crest.

Delivery and Miner's Inch Overflow Box of Azusa Irrigation Co., Southern California (Fig. 154).—This box differs from the previous one in that instead of delivering the water through an orifice adjustable in size by a slide, it is delivered through a number of openings, all 5 inches high but of different widths, each closed by a vertical sliding gate. The pressure on the center of opening is 4 inches, which is the commonly accepted head for the southern California miner's inch.

The water delivered through the orifices, discharges into a receiving basin, to which the irrigator connects his pipe line or flume. The basin may be a rectangular box built as a part of the overflow box, or may be formed of a section of large sized

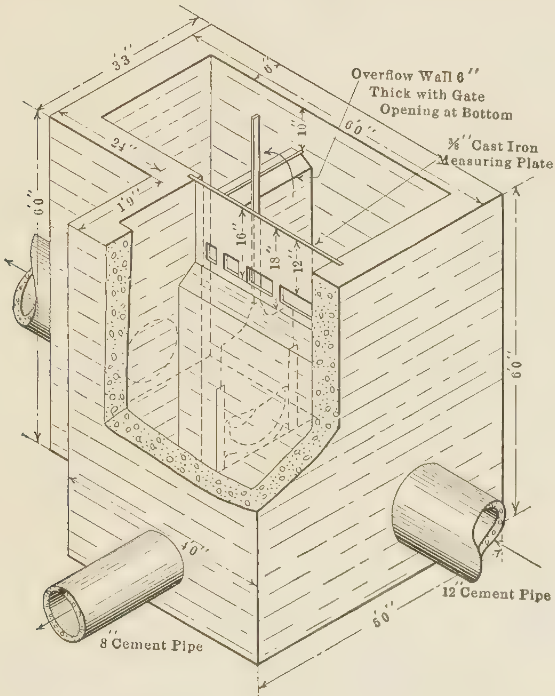


Fig. 154.—Delivery and miner's inch overflow box on pipe line. Azusa Irrigation Co., Southern California.

cement pipe, split longitudinally in two and cemented to the side wall of the overflow box (Plate XVIII, Fig. A). In the illustration the upper half section of pipe has not been put in place. The three smaller orifices are opened and the fourth closed.

CHAPTER XIII

MEASURING DEVICES

This chapter is concerned with the structures used on irrigation systems for the measurement of irrigation water. The principles of operation of the devices are presented and special emphasis is laid on the types of devices and the conditions for which each is best adapted. For a more complete discussion of the hydraulic laws, principles, experiments, formulæ and coefficients pertaining to the different types of weirs, the reader is referred to the standard books on hydraulics and to some of the references given at the end of the chapter.

Necessity for Measurement of Water.—The measurement of irrigation water by means of devices used specially for this purpose has been largely limited to those projects where the value of water, created by a demand in excess of the supply, has made it necessary to obtain a more nearly equal division of the water than could be obtained by the crude methods of operation and distribution prevailing on those systems which are favored with an excessive supply. On many systems where these crude methods prevail the division of the water between the laterals and distributaries is made by eye or by a rough measurement of the canal cross section with float measurements of the velocities, and the flow delivered to the water users is apportioned by an equally rough division of the flow in the distributary, or by a rough measurement taken at the delivery gate or, in the case of a distributary carrying a single irrigating head, by apportioning the entire head to each user in turn for a time proportionate to the acreage in his holding. While an experienced ditch tender may be able to make fairly close estimates of the flow of water in canals, these crude methods combined with the difference in experience and judgment of the ditch tenders will lead to unequal division with in some cases very great variations. As long as there is an abundant supply to satisfy all reasonable demands, and provided the water user pays for the use of water not according to the quantity used but on a flat rate per acre, there will be little friction or trouble between the water users and the officials

FIG. A.—Delivery and overflow miner's inch box for takeout from pipe line under no pressure. Azusa Irrigation Co., Southern California.

FIG. B.—Cippoletti weir board installed on small ditch.

(Facing page 374)

FIG. C.—Cippoletti weir with outlet wings and floor.

FIG. D.—Rectangular weir with end contractions.

of the water company, but a large waste and excessive use will generally result. These conditions are, however, seldom obtained, and there is practically no system designed of sufficient capacity to meet the abnormal demands. In the majority of systems there are periods of deficient flow and excessive demands when it is necessary to distribute the water in accordance with schedules which will produce equitable division.

With the development and extension of irrigation, in many localities the stage has been reached where the available water supply is not sufficient to supply all lands suitable for irrigation; it is then necessary to eliminate all preventable waste by a careful use of water, which can only be obtained by the introduction of a system of operation based on measurement of water. The importance of this is apparent when a careful study of the average irrigation practice indicates that of the water diverted from streams for irrigation 40 per cent. is lost by conveyance before it is delivered to the water user, and of the amount delivered 25 per cent. is lost by deep percolation, 25 per cent. by soil evaporation, and 10 per cent. by surface run-off. The total of these losses is about 76 per cent. of the water diverted, and by the best prevailing practice may be reduced to about 25 to 30 per cent.

The measurement of irrigation water is of special importance when the water user pays for the actual amount of water used; because the water user will then have just cause to dispute a water bill based on crude methods of measurement. On many systems irrigation water is paid for at a fixed rate per acre independent of the quantity used, but this practice is in many cases being replaced by payment for the volume of water used.

Locations on an Irrigation System where Measurement of Water is Necessary.—Measurements of the flow of water may be necessary at the following places on an irrigation system: (1) At the head of the diversion canal. (2) At the head of main canals, laterals, and distributaries. (3) At miscellaneous points on a canal lateral or distributary. (4) At points of delivery to the irrigators.

Measurements at the head of the diversion canal are necessary:

First.—To control the amount of water admitted in the canal system and insure that not more water is admitted than the system can carry.

Second.—To regulate the flow in accordance with the demand or supply.

Third.—To have the necessary information regarding the amount diverted in case of law-suits.

Fourth.—To study conveyance losses.

Measurements at the heads of laterals and distributaries and on the supply canal or lateral, from which these branches take out, within a short distance from the point of division are necessary:

First.—To divide the flow equitably.

Second.—To regulate the flow in accordance with demands.

Third.—To study conveyance losses. Measurements at different points on the canals, laterals and distributaries may be necessary to study and localize the conveyance losses in certain sections of canals.

Measurements at the points of deliveries are specially necessary when the water is charged for according to the volumes used. They are less necessary with a proper system of rotation, in which case the flow turned into the distributary may be a single irrigating head allotted in turn to each water user for a period of time proportional to the acreage of his holding or to the shares he owns or for the time required to give him the volume he wishes to purchase. Where the distributary carries two or more heads, measurements will be required to divide the flow.

Types of Measuring Devices.—The types of measuring devices may be divided into three classes:

First.—Those that give essentially the rate of flow; this includes: (1) rating stations and rating flumes; (2) weirs; (3) orifices, gate openings, and tubes or culverts.

Second.—Those that measure the quantity or volume of water in a given time; this includes the following types of meters: (1) the Venturi meter; (2) the Dethridge meter; (3) the Grant Michell meter; (4) the Hill meter; (5) the Hanna meter.

Third.—Some of the devices of the first type, usually rating stations or flumes and weirs, each combined with an automatic register with which volume as well as rate of flow can be obtained.

Measurements of the rate of flow are usually all that are necessary at the heads of canals, laterals and distributaries, and at different places on these parts of the system where measurement is necessary. These measurements are primarily to obtain proper operation of the system. At some of the above points it may be desirable to obtain also a continuous record of the flow, but when a canal system is properly operated the fluctuations in

water level in the different parts of the system will not be very great and readings of the rate of flow once or twice daily will be sufficient.

Measurements of the actual volume of water in a given time are generally only necessary or desirable at the points of deliveries when water is charged for on the basis of the water used. The types of meters associated with this method of measurement have been used only to a small extent, but will probably be used to a greater extent with the establishment of this method of payment for irrigation water and the increase in the value of water.

In the past rate of flow measurements have been largely depended upon and with good operation of the canal system give results whose accuracy is consistent with the present value of water in many localities.

Conditions Controlling the Use of Different Devices.—The selection of the type of measuring device will depend on the topographic conditions, the character of the irrigation water, and a number of desirable requirements. Where the irrigable land is flat valley land, the measurement of water must usually be made with devices which require a minimum fall or loss of head for their operation. Where the irrigation water carries much silt no device should be used which will cause the deposit of this silt in the canals and interfere with the measurement. Where the irrigation transports weeds or other large material the device should not be easily obstructed. Other desirable requirements are the following:

First.—The device should not be easily tampered with.

Second.—The cost of the device should not be excessive.

Third.—It should preferably measure not only the rate of flow, but also register the volume of water in a given time.

Fourth.—In most cases it should be able to handle any fraction of its full capacity.

Fifth.—In most cases it is desirable that when once set for the desired capacity, it maintains a constant flow, or a flow which is not affected to a large extent by variations in water levels.

RATING STATION AND RATING FLUME

A rating station consists of a selected section on the canal whose discharge is known for any depth of water, usually indicated on a permanent gauge rod.

To obtain this relation between depth of water and corresponding discharge, it is necessary to rate the station. This is done by taking a series of measurements which will give the discharges corresponding to depths of water, included in the range commonly used in the operation of the canal. The results are plotted and a curve drawn through the discharge points. This curve is known as the rating curve; with it the discharge corresponding to any depth of water can be obtained, and from it a rating table may be prepared. The discharge measurements required to obtain the rating curve when using a current meter are made as follows: The cross-sectional area of the station is first divided by imaginary vertical lines into partial or elementary areas whose cross section is obtained by soundings, taken usually with a rod, or, if rod current meter is used, with the current meter itself (Fig. 155). The number of verticals to be used in dividing the cross-sectional area will depend on the average width of the cross section and on the form of the cross section. It will be less for

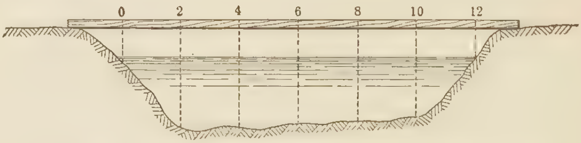


Fig. 155.—Rating station divided into partial areas.

a rectangular flume section than for an earth canal section. Usually at least six to eight verticals are desirable. The average velocity for each elementary area is then obtained and the corresponding discharge is the product of this velocity and the elementary area. The total discharge is the sum of the partial discharges.

The current meter with which the velocities are obtained consists of a wheel with cups which, when placed in water, gives a rate of revolutions proportionate to the velocity of flow. The number of revolutions in a given time are obtained through a sounding device attached to the meter and the relations between rate of revolutions and velocity are known from experimental rating of the meter. For a full presentation of the use of current meters and the principles and methods of measurement of flow, "River Discharge," by Hoyt and Grover, published by Wiley and Sons, is specially recommended. The methods of measurements are essentially the same for canal flow as for river discharge.

The most common methods of obtaining the mean velocity in a vertical line with a current meter are: (1) by holding the current meter at 0.2 and at 0.8 of the water depth and taking the mean of the two corresponding velocities; (2) by holding the meter at 0.6 of the water depth; (3) by moving the current meter slowly down, then up the water depth and repeating one or more times. The first method is usually considered preferable. The second method gives slightly greater results; it is more often used for shallow depths.

Measurements by floats require a stretch of canal with uniform cross section of at least 100 feet in which the velocity of a float is obtained by taking the time required to travel between two stations. The floats do not often travel in straight lines; it is therefore difficult to obtain reliable velocities for the different parts of the stream, and the mean velocities of flow can only be obtained by applying certain coefficients of doubtful accuracy.

Method of Installation.—The accuracy of measurements obtained at a station which has been rated will depend on the permanency of its cross section, the conditions of flow and the factors which may affect the accuracy of the rating curve. In an earth canal it is sometimes difficult to obtain a site where the cross section will not be changed by erosion, deposition of silt or growth of vegetation. To obtain a permanent station it is often necessary or desirable to line a short section of the canal with concrete or to install a rating flume. A rating flume consists of a short flume, whose length is generally not less than 12 feet nor less than twice the average width of the canal or flume, and whose width and depth are about equal to the average width and depth of the canal. The inlet and outlet are connected to the earth canal with suitable wings and cut-off walls (Fig. 156). Transversally the floor should be perfectly level and longitudinally it should be set to the grade of the ditch. The floor should be placed about $\frac{1}{10}$ of a foot above the bed of the ditch, so that all silt may be carried through and not change the form of the cross section. The gauge rod should be placed at a distance from the upstream end equal to about $\frac{3}{4}$ of the length of the flume. To obtain a continuous record of the stream flow, as is often desirable at important rating stations or flumes, an automatic register is necessary. The common types of registers are described below. The use of a register requires a float well, usually 12×18 inches in cross section. For a canal station

the well is placed in the canal bank and connected to the canal by means of a short pipe. For a rating flume, the float well is built with the flume, on one side, and connected with the water by an opening or a number of smaller orifices through the side wall near the bottom of the flume.

To obtain accurate results, a rating station or flume must be selected, so as to obtain a uniform flow free from cross currents and located sufficiently far upstream from check gates or delivery gates to be beyond the influence of backwater or drawdown produced by the operation of check gates, lateral headgates and

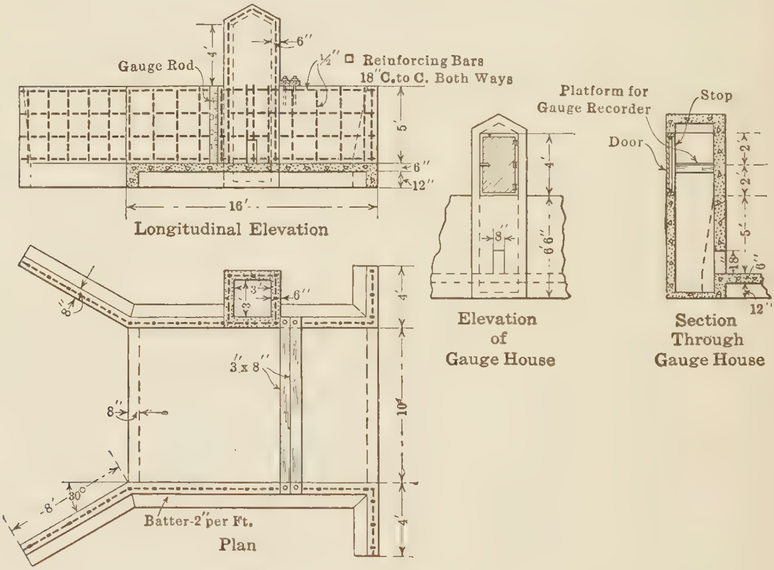


FIG. 156.—Rating flume. Colo. Fuel & Iron Co., Pueblo, Colo.

delivery gates. To obtain a uniform flow, a straight channel with uniform cross section and smooth banks for a distance of at least 100 feet upstream from the station or flume is necessary. The distance upstream from check gates, lateral or delivery gates required to be beyond the effect of backwater or drawdown will depend on a number of factors, such as the slope and size of the canal, the height of checking, the capacity of the delivery gate, etc. To be beyond any appreciable effect may require a distance of several thousand feet for a large canal. The best location for a rating station or flume will therefore be above a drop not used as a check gate. Other serious causes which may destroy

the rating are the deposition of silt, the erosion, and the growth of vegetation in the canal during the irrigation season.

The main advantage of rating stations or flumes is their adaptation to the measurement of large flows with no loss of head or velocity. The most serious disadvantages indicated above are that they should be located far upstream from check gates, headgates and delivery gates, and that the ratings will be put in error by the growth of vegetation, deposition of silt, and erosion.

WEIRS

Types of Weirs.—The term weir is applied to any dam or barrier across a stream over which water flows. The weirs commonly used for measuring devices are of three types: (1) The Cippoletti or trapezoidal weir; (2) the rectangular weir with end contractions; (3) the rectangular weir with suppressed end contractions.

The Cippoletti or trapezoidal weir is a trapezoidal notch in the top of a board, formed of a horizontal crest and of two sides, both sloping outward 1 inch for every 4 inches of rise (Plate XVIII, Figs. B and C and Plate XIX, Figs. A and B.)

The rectangular weir with end contractions is a rectangular notch in which the vertical sides are sufficiently far removed from the sides of the channel or flume box in which it is placed to fully contract the issuing jet of water on the sides (Plate XVIII, Fig. D.)

The rectangular weir with suppressed end contractions is formed of a vertical board, not notched, placed in a channel formed between two vertical side walls, such that the water will flow over the overpour horizontal crest of the board for its entire length between side walls with no end contractions.

The triangular or V-shaped notch weir is seldom used on irrigation systems.

Weirs and submerged orifices are the most common types of measuring devices used on irrigation systems. Weirs are simple to construct, install and use, and will give accurate results when properly installed under conditions favorable to their use. Their use is most commonly limited to moderate quantities of water and where the required fall for their installation is obtainable. Water carrying much silt will usually preclude their use.

The measurement of the rate of flow over the weir is a very simple operation. Usually the weir is installed so that the velocity

of the water in the upstream channel is so small that this velocity of approach can be neglected. The only measurements required are, then, that of the length of the weir and that of the depth of water over the weir. The corresponding flow may be computed with the weir formula, but is generally obtained by referring to tables.

The weir most commonly used is the Cippoletti weir; it has the advantage that the flow is proportional to the length of the weir, so that a weir table prepared for a unit length of weir may be used for other lengths by multiplying the quantities given in the table by the ratio of the desired length to unity.

Method of Installation and Use.—A weir measuring device may consist of a simple weir board placed across the ditch (Plate

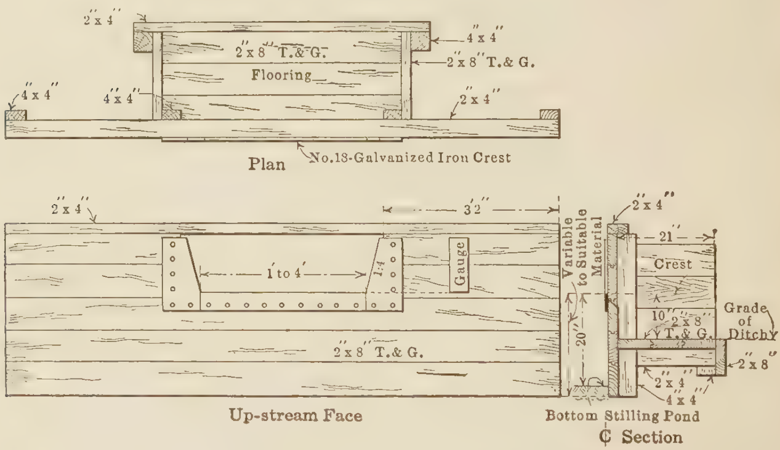


Fig. 157.—Standard design for Cippoletti weirs on system of Twin Falls Salmon River Water Co., Idaho.

XVIII, Fig. B) or of a weir board set in a box, or short flume section which may be built as an extension of a delivery gate structure. The weir board may be made of wood, metal or concrete. When made of wood or concrete it is desirable to use a metal plate to form the edges of the crest and the sides. When the device is a simple weir board, placed in the earth ditch, the board must extend sufficiently far into the bed and sides of the ditch, and must be well puddled in position to prevent the water washing under or around the board. Large weir boards must be braced with posts against the water pressure, and will usually require protection of the bed and sides of the ditch on the outlet side of the

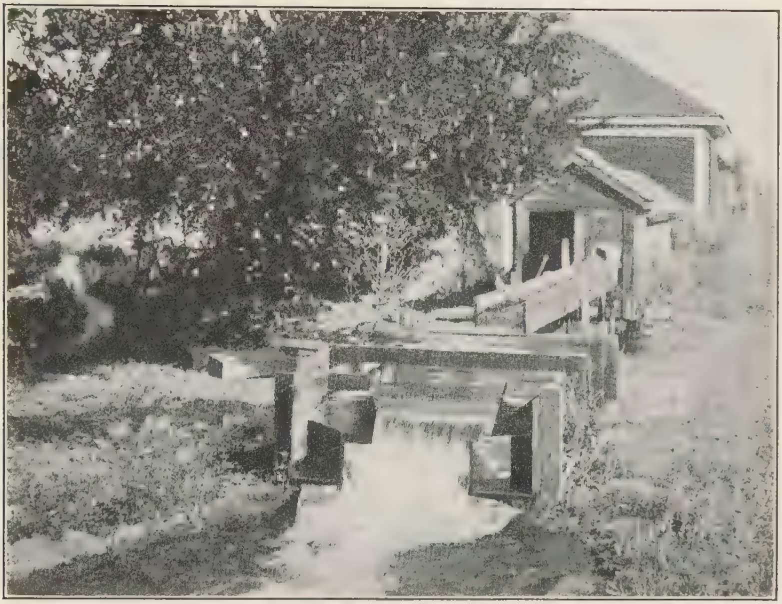


FIG. A.—Cippoletti weir box to measure discharge of pumping plant.

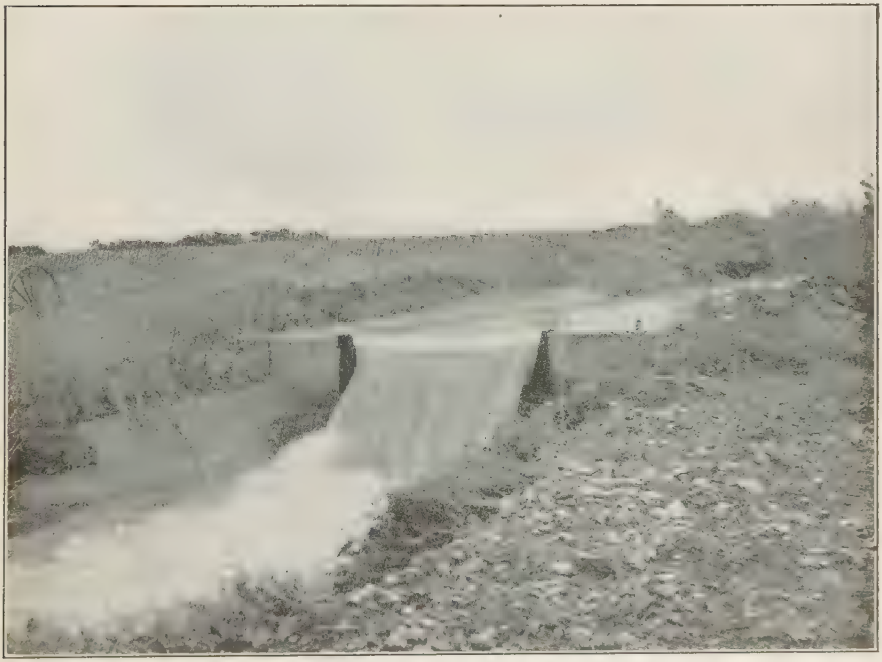


FIG. B.—Cippoletti weir in concrete lined section of lateral. Davis and Weber Canal System, Utah.

(Facing page 382)

board, against the erosive force of the overpour water. A common form of construction consists of the addition of an outlet extension, formed of a floor between two side walls (Figs. 157 and 158). The ditch cross section upstream from the weir board must be sufficiently large to form a stilling basin in which the velocity of approach is low.

When the weir is placed in a flume box the width and depth of the box must be at least sufficient to give the required dimensions

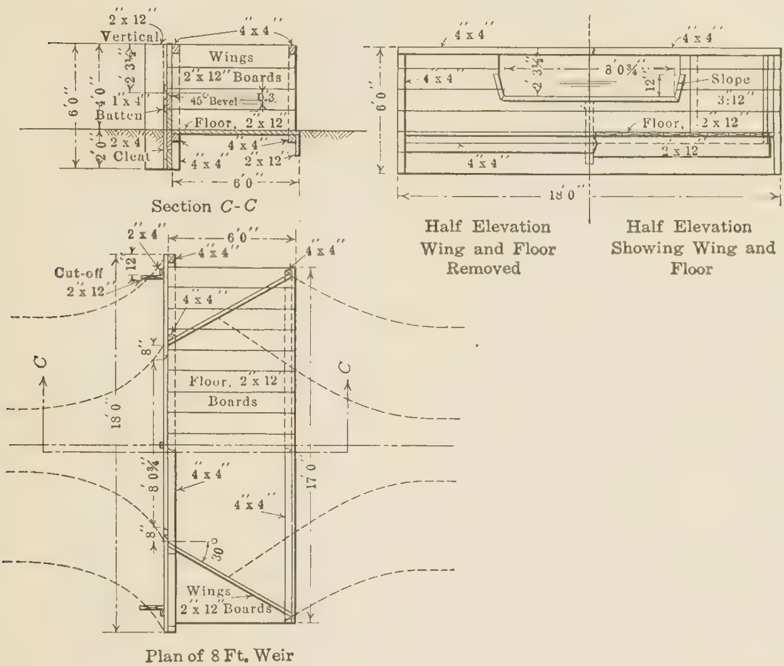


FIG. 158.—Cippoletti weir (weir plate not shown). Yakima Sunnyside Project, Wash.

of the weir board as specified by the requirements stated below (Fig. 159). The length of the flume box is generally made about 2 or 3 times its width, and the weir board is placed at a distance from the upstream equal to about ⅔ the length of the box, the lower ⅓ forming the protection against the overpour water. The box is connected to the ditch at the inlet and outlet with wings and cut-off wall. The use of a flume box is necessary for the installation of a weir with suppressed contraction; when used for a Cippoletti weir or a rectangular weir with

end contraction, it gives no advantage over the simpler form consisting of a weir board with outlet extension and increases the cost.

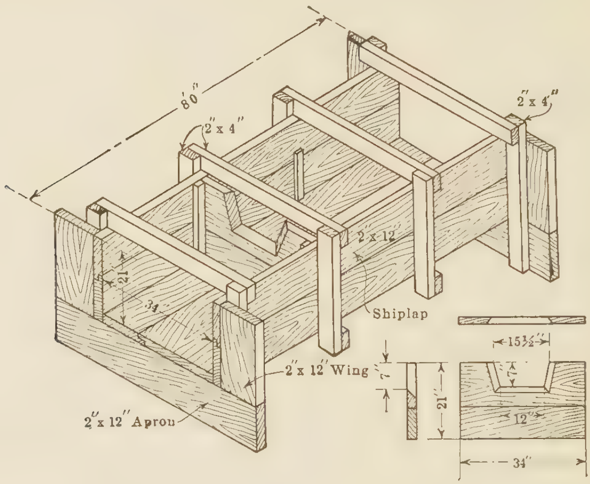


FIG. 159.—Cippoletti weir box for 1-ft. weir.

A suppressed weir requires some means of providing free circulation of air under the sheet of falling water; this is usually ob-

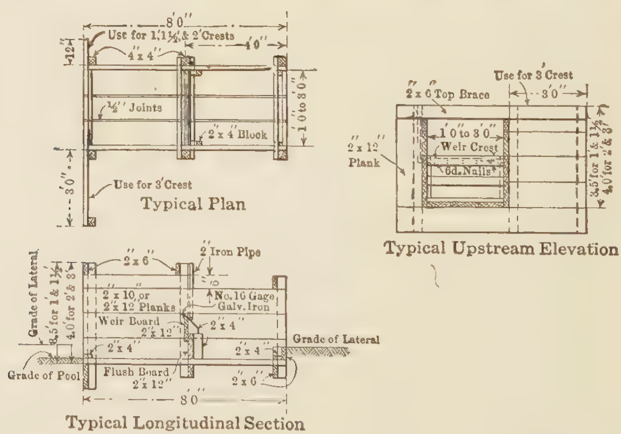


FIG. 160.—Standard suppressed rectangular weirs. U. S. Reclamation Service.

tained by admitting the air through an opening in one or both side walls near the crest on the downstream side (Fig. 160).

When the weir is placed a short distance below a delivery gate,

the weir board is placed in a short flume section or stilling box built as an extension to the gate structure (Fig. 161). It is then

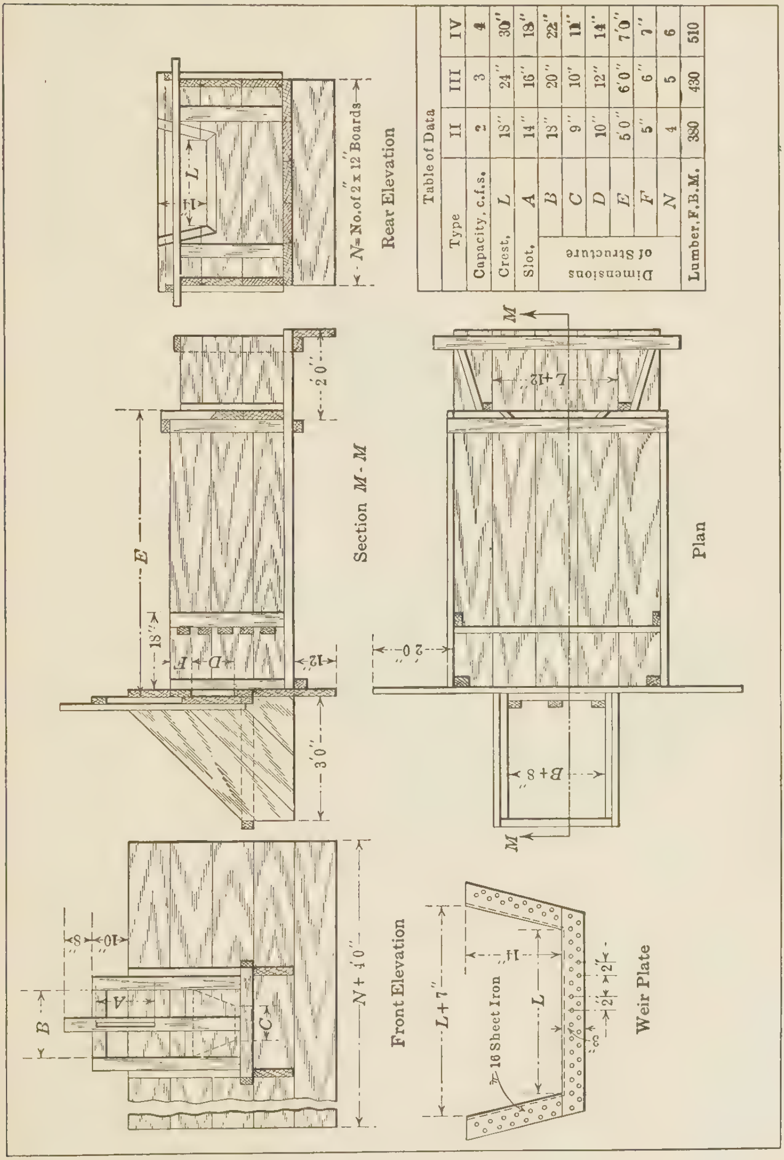


Table of Data

Type		II	III	IV
Capacity, c.f.s.		2	3	4
Crest,	L	18"	24"	30"
Slot,	A	14"	16"	18"
Dimensions of Structure	B	18"	20"	22"
	C	9"	10"	11"
	D	10"	12"	14"
	E	5'0"	6'0"	7'0"
	F	5"	6"	7"
	N	4	5	6
Lumber, F.B.M.		380	430	510

FIG. 161.—Delivery gate and weir box. Yakima Project, Tieton Unit, Wash.

usually necessary to provide baffles to produce a uniform flow before the water reaches the weir board.

The measurement of the depth of overpour water over the weir must show the true elevation of the water surface above the weir crest. Directly at the crest and for a short distance above it the water surface curves down. This requires that the measurement for the depth of water be taken in the still water on one side of the weir notch or for a relatively small weir, from 4 to 6 feet upstream from the weir board. Experiments on Cippoletti weirs made in Punjab, India, indicate that at a distance of 1 foot to one side of the edge of the notch the water level was not affected by the draw of the water toward the notch. The point of depth

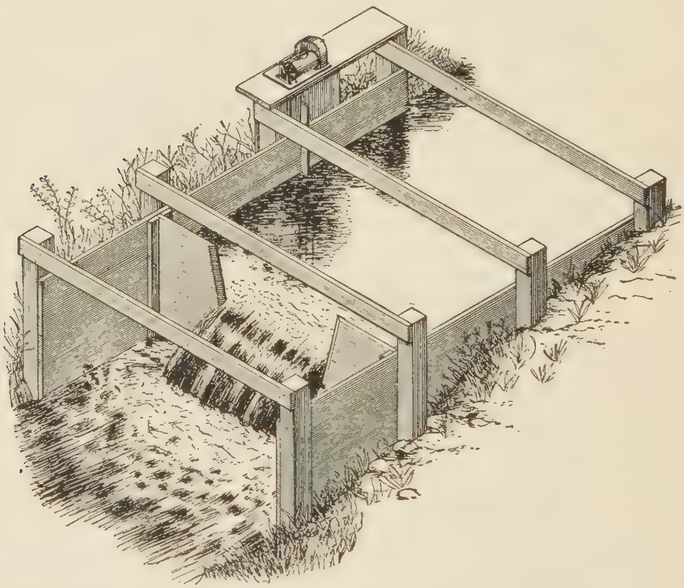


FIG. 162.—Isometric sketch of Cippoletti weir box with automatic register.

measurement or gauge is often placed in this position on the weir board at one side of the notch. When a continuous record of the flow is desired, an automatic register must be used. This is placed at the top of a float well built on one side of the flume box or set in the bank or side of the ditch upstream from the weir board (Fig. 162).

To sluice out silt deposited in the stilling basin above the weir board it is desirable to make the entire weir board removable, or preferably to make only the lower part removable, which insures that the position of the weir crest is not changed.

Weir Dimensions and Requirements of Installation for Accurate Results.—To obtain accurate results the following rules should control the dimensions of the notch:

1. The depth of water allowed on the crest of the weir should be not greater than $\frac{1}{3}$ the length of the weir and not smaller than 1 inch. Within these limits the length of the weir and depth of overpour is generally determined by the fall available. Where the fall available is small, a large length and small depth will be used.

This requirement is usually specified as necessary because the weir formulæ are the results of experiments made within the above limits of head. But recent experiments made by W. G. Steward and J. S. Longwell on Cippoletti weirs and rectangular weirs with end contractions, 6, 12, 24 and 36 inches in length, indicate that for a head as great as $\frac{1}{2}$ the length, the actual flow is less than 3 per cent. greater than the computed flow for Cippoletti weirs and less than 6 per cent. greater for rectangular weirs. Their results, however, show that for even smaller heads, less than $\frac{1}{3}$ the length, the actual flow with rectangular weirs is from 2 to 3 per cent. greater than the computed flow with Francis formula, while for Cippoletti weirs the actual and computed flows are equal.

2. The distance from the crest of the weir to the bed of the canal or floor of the weir box on the upstream side of the weir should be at least twice and preferably 3 times the depth of water on the weir. This is necessary to produce full bottom contraction.

3. The distance from the edges of the weir notch to the sides of the canal or of the weir box should be at least twice the depth of water on the weir. This is necessary to produce full side contractions.

4. The upstream edge of the crest and sides of the notch should be a sharp square corner or brought to a knife edge, either by using a thin metal plate or by bevelling on the downstream side. With a sharp corner and a thickness not greater than half the minimum depth of water, the discharge will be the same as for a knife edge. The importance of a sharp corner or crest to form the upstream edges of the crest and sides of the notch is indicated by experiments made by J. C. Stevens on the Sunnyside project, Washington. The tests were made on Cippoletti weirs, 6 inches to 3 feet in length. The notches were made in a 2-inch plank,

with the edges bevelled to a 45° slope on the downstream side and covered with No. 24 galvanized iron plate. This formed nearly sharp edges, with a radius of curvature only a little greater than the thickness of the galvanized iron (about 0.02 inch), yet it appeared to have been sufficient to make the actual discharges about 5 per cent. greater than the computed discharge.

The above requirements will insure a comparatively small velocity of approach, which can be neglected without producing an appreciable error. When the water cross-sectional area of the notch is $\frac{1}{5}$ that of the channel, the error introduced by neglecting the velocity of approach is about 2 per cent. The requirements (2) and (3) insure a ratio of cross-sectional areas of at least $\frac{1}{7}$.

For a weir without end contractions the requirements, excepting those regarding side contractions, are essentially the same when the discharge is obtained from computations or tables based on the general accepted Francis formula. However, the exhaustive study of weirs without end contraction, recently made by Prof. R. R. Lyman, has resulted in the presentation by him of diagrams and tables which will permit the accurate measurement of water with weirs without end contractions, for heights of weir less than that prescribed by the above requirements. The table, reproduced in part further, takes into account the velocity of approach, which with low weirs and large depths of water is considerable.

The accuracy of weir measurements will also depend on the extent to which the following installation requirements are met:

1. The weir when installed on a ditch should be placed in a section of ditch which is straight for at least 50 feet upstream from the weir and the center line of the ditch should be perpendicular to the weir board and pass through its center. The cross section of the channel should not be smaller than the cross section of the weir box or board in order to have slow velocity and fairly calm water above the weir. If the weir board must be placed near a takeout gate, the velocity of approach must be made uniform by means of baffles.

2. The weir must be set high enough to give to the overflowing sheet a free fall. However, where the required fall for this is not obtainable, the downstream water level may be above the weir crest as high as 15 per cent. of the depth of water on the crest

without introducing an excessive error. With greater submergence the weir can still be used and give fairly accurate results by the application of certain coefficients.

3. In letting water in a weir box through a pipe, it should discharge preferably at the bottom of the box, and the depth of the box should be sufficient to produce a calm body of water on the upstream side of the weir. In some cases this requires the use of baffle boards to break up the velocity of the approaching water.

4. The crest of the weir should be level from end to end, and the weir board placed vertically. The importance of having the weir board placed exactly vertical is not great, and is shown by the following results of experiments made by Bazin on thin-edged weirs inclined at various angles.

Relative Discharge of Inclined Weirs and Vertical Weirs by Bazin's Experiments

		Bazin's modulus
Vertical weir		1.00
Upstream inclination of the weir	1 horizontal to 1 vertical	0.93
	2 horizontal to 3 vertical	0.94
	1 horizontal to 3 vertical	0.96
Downstream inclination of the weir	1 horizontal to 3 vertical	1.04
	2 horizontal to 3 vertical	1.07
	1 horizontal to 1 vertical	1.10
	2 horizontal to 1 vertical	1.12
	4 horizontal to 1 vertical	1.09

Formulæ and Tables for Measurement of Flow of Water Over Weirs.—A number of formulæ have been derived; those most commonly used are the following:

I. Formulæ for free discharge when velocity of approach is negligible.

(*a*) Rectangular weir with end contractions (Francis formula),

$$Q = 3.33\ (L - 0.2H)H^{3/2}$$

(*b*) Rectangular weir with no end contractions (Francis formula),

$$Q = 3.33LH^{3/2}$$

(*c*) Cippoletti weir,

$$Q = 3.367\ LH^{3/2}$$

II. Formulæ for free discharge when velocity of approach must be considered.

(*d*) Rectangular weir with end contractions (Francis formula),

$Q = 3.33(L - 0.2H)[(H + h)^{3/2} - h^{3/2}]$

(*e*) Rectangular weir without end contractions (Francis formula),

$Q = 3.33L[(H + h)^{3/2} - h^{3/2}]$

(*f*) Cippoletti weir,

$Q = 3.367L(H + 1.5h)^{3/2}$

In the above formulæ the following notation has been used:

Q = rate of flow over weir in cubic feet per second.
L = length of weir crest in feet.
H = head or depth of overpour on the crest in feet, measured where not affected by drawdown curve.
h = velocity head corresponding to velocity of approach v, and equal to $\frac{v^2}{2g}$

III. Formulæ for submerged discharge.

When the water level on the downstream water level rises above the weir crest, the weir is said to be submerged. As previously stated, a submergence of as much as 15 per cent. or even 20 per cent. will produce very little effect on the flow. Clemens Herschel, from a study of experiments on weirs without end contractions made by J. B. Francis and Fteley and Stearns has derived the following formula:

$$Q_1 = 3.33L(NH)^{3/2}$$

in which N is a coefficient whose value depends on the proportional submergence $\frac{d}{H}$ where d is the depth of submergence.

This formula gives the following relation:

$$\frac{Q_1}{Q} = N^{3/2} \text{ or } Q_1 = N^{3/2}Q$$

in which for equal head (H) and equal lengths of weir Q_1 is the rate of flow for a submerged weir and Q is the rate of flow for a free discharge weir. Therefore the rate of flow over a submerged weir is equal to the rate of flow over a free discharge weir of the same length and under the same head multiplied by a coefficient ($N^{3/2}$).

While the above formula was derived for a rectangular weir with no end contractions, experiments made by J. C. Stevens on

the Sunnyside project in Washington and by A. S. Gibb in Punjab, India, on Cippoletti weirs give values of N very nearly equal to those given by Clemens Herschel. These values of N are tabulated below.

VALUES OF MULTIPLIER (N) TO BE APPLIED TO THE HEAD (H) ON THE WEIR CREST OF A SUBMERGED WEIR TO OBTAIN THE EQUIVALENT HEAD FOR THE SAME WEIR WITH FREE FALL; ALSO VALUES OF MULTIPLIER ($N^{3/2}$) TO BE APPLIED TO THE DISCHARGE OF A WEIR WITH FREE FALL TO OBTAIN THE DISCHARGE OF SAME WEIR WHEN SUBMERGED

Percentage of submersion	Weirs without end contraction by Clemens Herschel table		Cippoletti weirs			
			By J. C. Stevens		By A. S. Gibb	
	N	$N^{3/2}$	N	$N^{3/2}$	N	$N^{3/2}$
5	1.007	1.0105	....	....	0.994	0.991
10	1.005	1.0075	0.985	0.977	0.985	0.977
15	0.996	0.994	0.98	0.970	0.975	0.963
20	0.985	0.978	0.97	0.955	0.964	0.946
25	0.972	0.958	0.96	0.941	0.952	0.929
30	0.959	0.939	0.95	0.926	0.939	0.911
40	0.929	0.897	0.93	0.897	0.910	0.868
50	0.892	0.842	0.90	0.854	0.877	0.821
60	0.846	0.778	0.85	0.784	0.840	0.770
70	0.787	0.689	0.80	0.715	0.799	0.715

Tables of Weir Discharge.—The following tables are abstracted from more extensive tables presented in a number of publications.

The table for the Cippoletti weir gives the rates of discharge for a 1-foot length of weir; for other lengths of weir the discharge is obtained by multiplying the values given by the length of the weir in feet.

The table for the rectangular weir with end contractions is prepared for weir lengths of 1, 1½, 2 and 3 feet, which are commonly used in practice.

The table for the weir without end contraction is abstracted from much more extensive tables prepared by Prof. Richard R. Lyman, of the University of Utah. These tables result from a careful and exhaustive study by Prof. Lyman of the results of experiments made by Bazin, Francis, Fteley and Stearns and at the laboratories of Cornell University and the University of Utah. The table gives the rates of discharge of a 1-foot length of weir for different heights of weir crest above the floor of the box

in which the weir board is placed. The velocity of approach is largely controlled by the height of the weir, and its effect is included in the values given. The results of the work of Prof. Lyman make it possible to obtain accurate measurements with weirs without end contractions, where the high velocity of approach and relatively large depth of water on the crest would result in errors of considerable magnitude if their use was combined with the usually accepted Francis formula.

Discharge Table for Cippoletti Weir, 1 Foot in Length
(For other lengths multiply values given by length in feet)

Depth of water on crest		Discharge, cubic feet per second	Depth of water on crest		Discharge, cubic feet per second	Depth of water on crest		Discharge, cubic feet per second
In inches	In feet		In inches	In feet		In inches	In feet	
1	0.08	0.08	3⅝	0.30	0.56	6¾	0.56	1.42
1⅛	0.09	0.10	3¾	0.31	0.59	7	0.58	1.50
1¼	0.10	0.11	3⅞	0.32	0.62	7¼	0.60	1.58
1⅜	0.11	0.13	4	0.33	0.65	7½	0.62	1.66
1½	0.12	0.15	4⅛	0.34	0.68	7¾	0.65	1.75
1⅝	0.14	0.17	4¼	0.35	0.71	8	0.67	1.83
1¾	0.15	0.19	4⅜	0.36	0.74	8¼	0.69	1.92
1⅞	0.16	0.21	4½	0.37	0.77	8½	0.71	2.07
2	0.17	0.23	4⅝	0.39	0.81	8¾	0.73	2.10
2⅛	0.18	0.25	4¾	0.40	0.84	9	0.75	2.19
2¼	0.19	0.27	4⅞	0.41	0.87	9¼	0.77	2.28
2⅜	0.20	0.30	5	0.42	0.91	9½	0.79	2.37
2½	0.21	0.32	5⅛	0.43	0.94	9¾	0.81	2.47
2⅝	0.22	0.34	5¼	0.44	0.97	10	0.83	2.56
2¾	0.23	0.37	5⅜	0.45	1.01	10¼	0.85	2.66
2⅞	0.24	0.40	5½	0.46	1.04	10½	0.87	2.76
3	0.25	0.42	5⅝	0.47	1.08	10¾	0.90	2.85
3⅛	0.26	0.45	5¾	0.48	1.12	11	0.92	2.95
3¼	0.27	0.47	5⅞	0.49	1.15	11¼	0.94	3.06
3⅜	0.28	0.50	6	0.50	1.19	11½	0.96	3.16
3½	0.29	0.53	6¼	0.52	1.26	11¾	0.98	3.26
......	......	....	6½	0.54	1.34	12	1.00	3.37

For a 1-foot weir use depths of water not greater than 4 inches.
For a 1½-foot weir use depths of water not greater than 6 inches.
For a 2-foot weir use depths of water not greater than 8 inches.
For a 3-foot weir use depths of water not greater than 12 inches.

Discharge Table for Rectangular Weirs with Full Contractions

Depth of water on crest		Discharge in cubic feet per second for			
In inches	In feet	1-foot weir	1½-foot weir	2-foot weir	3-foot weir
1	0.08	0.079	0.119	0.159	0.239
1⅛	0.09	0.094	0.142	0.189	0.285
1¼	0.10	0.110	0.166	0.222	0.33
1⅜	0.11	0.126	0.191	0.255	0.38
1½	0.12	0.144	0.217	0.29	0.44
1⅝	0.14	0.161	0.244	0.32	0.49
1¾	0.15	0.180	0.273	0.36	0.55
1⅞	0.16	0.20	0.302	0.40	0.61
2	0.17	0.22	0.332	0.45	0.67
2⅛	0.18	0.24	0.364	0.49	0.74
2¼	0.19	0.26	0.395	0.53	0.80
2⅜	0.20	0.28	0.428	0.58	0.87
2½	0.21	0.30	0.462	0.62	0.94
2⅝	0.22	0.32	0.496	0.67	1.01
2¾	0.23	0.35	0.531	0.72	1.08
2⅞	0.24	0.37	0.567	0.76	1.16
3	0.25	0.40	0.604	0.81	1.23
3¼	0.27	0.44	0.679	0.91	1.39
3½	0.29	0.49	0.756	1.02	1.54
3¾	0.31	0.54	0.836	1.13	1.71
4	0.33	0.60	0.919	1.24	1.88
4¼	0.35	0.65	1.003	1.36	2.06
4½	0.37	0.71	1.090	1.47	2.24
4¾	0.40	0.76	1.178	1.59	2.43
5	0.42	0.82	1.269	1.72	2.61
5¼	0.44	0.86	1.361	1.84	2.81
5½	0.46	0.94	1.455	1.97	3.00
5¾	0.48	1.00	1.550	2.11	3.20
6	0.50	1.06	1.649	2.23	3.41
6¼	0.52	....	1.747	2.38	3.63
6½	0.54	....	1.848	2.51	3.87
6¾	0.56	....	1.949	2.65	4.06
7	0.58	....	2.053	2.80	4.29
7½	0.62	....	2.263	3.00	4.47
8	0.67	....	2.477	3.40	5.20
8½	0.71	....	2.697	3.70	5.65
9	0.75	....	2.919	4.00	6.15
9½	0.79	....	....	4.30	6.64
10	0.83	....	....	4.64	7.15
10½	0.87	....	....	5.00	7.71
11	0.92	....	....	5.32	8.24
11½	0.96	....	....	5.65	8.78
12	1.00	....	....	6.00	9.34

Discharge Table for Rectangular Weir with no End Contractions for 1 Foot Length of Weir and Different Heights of Weir Crest
By R. R. Lyman

(For any length of weir the discharge is obtained by multiplying the values given by the length of weir in feet)

Head in inches	Head in feet	Weir 0.5 feet high	Weir 0.75 feet high	Weir 1 foot high	Weir 2 feet high	Weir 4 feet high
2⅜	0.200	0.315	0.314	0.313	0.311	0.309
2½	0.210	0.340	0.337	0.336	0.334	0.332
2⅝	0.220	0.365	0.363	0.360	0.357	0.355
2¾	0.230	0.392	0.388	0.385	0.382	0.380
2⅞	0.240	0.420	0.415	0.412	0.406	0.404
3	0.250	0.446	0.442	0.438	0.434	0.430
3¼	0.270	0.503	0.497	0.493	0.486	0.483
3½	0.290	0.560	0.552	0.547	0.540	0.533
3¾	0.314	0.640	0.627	0.620	0.608	0.602
4	0.335	0.705	0.690	0.680	0.665	0.657
4¼	0.355	0.770	0.752	0.743	0.725	0.717
4½	0.375	0.840	0.817	0.805	0.790	0.777
4¾	0.395	0.910	0.885	0.870	0.852	0.838
5	0.415	0.990	0.956	0.943	0.917	0.903
5¼	0.440	1.083	1.045	1.026	1.000	0.985
5½	0.460	1.164	1.125	1.105	1.074	1.050
5¾	0.480	1.250	1.205	1.185	1.150	1.125
6	0.500	1.335	1.285	1.263	1.220	1.195
6¼	0.520	1.415	1.360	1.335	1.290	1.260
6½	0.540	1.510	1.440	1.415	1.365	1.336
6¾	0.565	1.616	1.545	1.515	1.455	1.420
7	0.585	1.713	1.635	1.605	1.540	1.505
7½	0.625	1.905	1.815	1.780	1.705	1.670
8	0.665	2.110	2.005	1.965	1.880	1.830
8½	0.710	2.350	2.220	2.170	2.085	2.020
9	0.750	2.585	2.430	2.375	2.260	2.190
9½	0.790	2.820	2.630	2.570	2.430	2.360
10	0.835	3.100	2.905	2.830	2.675	2.580
10½	0.875	3.350	3.120	3.035	2.870	2.765
11	0.915	3.620	3.360	3.260	3.085	2.955
11½	0.960	3.940	3.640	3.540	3.325	3.190
12	1.000	4.230	3.900	3.780	3.555	3.400

ORIFICES, GATE OPENINGS AND SHORT TUBES

Many forms of openings are used under widely different conditions for the measurement of irrigation water. In all cases the opening is formed in a vertical plane and is placed below the upstream or inlet water level, and the measurement of the rate of flow is dependent on the area of the opening and the effective head which produces the velocity.

In order to obtain the rate of flow through any form of opening without the use of complicated formulæ, the measuring device should be installed so that the opening will be placed either entirely above the outlet water level, or entirely below the outlet water level. The many forms of these measuring devices may therefore be divided into two classes:

First.—Free discharge openings.

Second.—Submerged discharge openings. Each class may be subdivided into orifices of a fixed size and adjustable openings.

The approximate formula for rate of discharge is then:

$$Q = CA\sqrt{2gH}$$

in which

Q = rate of flow in cubic feet per second.
C = coefficient of discharge.
A = the area of the opening in square feet.
H = the effective head in feet, which in the case of free discharge is usually taken as the depth from the inlet water level to the center of the opening, and in the case of submerged discharge as the difference in elevation between the inlet and outlet water levels.

This formula does not consider the velocity of approach. In Vol. II, Chapter III, a formula is given which may be used when the velocity of approach is of sufficient magnitude to be considered and another formula is given for free discharge through large rectangular orifices under small heads. The use of these more accurate formulæ may in some cases be necessary, but for common use in irrigation work the simpler approximate formula is generally accepted. The accuracy of the results will depend largely on the form of the opening and the conditions of flow; these will be indicated in the discussion of the different types of device.

One advantage of an orifice device over a weir device, which may be stated here, is that a variation in the head on an orifice will produce a correspondingly smaller variation in the rate of flow through the orifice than an equal proportionate variation in depth of water over a weir will produce in the rate of flow over the weir. A 10 per cent. increase in the head will increase the flow through the orifice only 5 per cent., while with a weir the flow is increased 15 per cent.

Free Discharge Openings. Miner's Inch Measurement.—It is seldom possible to obtain sufficient fall to obtain free discharge except for smaller openings, such as are commonly associated with devices used for the delivery and measurement of water in miner's inches.

A miner's inch device is formed of a miner's inch board placed directly in the ditch or across the stream to be measured or installed in a box or structure. The miner's inch board contains either a single long opening, usually regulated in size by a slide or a number of openings of different sizes, each closed by a gate. The delivery through the orifices is made by maintaining a constant head on the center of the orifice. The head depends on the value of the miner's inch, which varies with different states (see Chapter V, Vol. I, pages 70 to 72). In Arizona, Montana, and Oregon, 40 miner's inches are equivalent to 1 cubic foot per second; this requires a 6-inch pressure head. In Idaho, Nebraska, Nevada, New Mexico, North Dakota, South Dakota and Utah, 50 miner's inches are equivalent to 1 cubic foot per second; this requires a 4-inch pressure head. In California, as defined by statute, 40 miner's inches are equivalent to 1 cubic foot per second, but on a number of systems in southern California 50 miner's inches to 1 cubic foot per second are used.

In some states the form of the orifice is specified by law. For instance in Colorado a miner's inch is defined by laws of 1868 as the flow through an inch orifice under a 5-inch pressure measured from the top of the orifice, and the prescribed height of the orifice is 6 inches except for flows under 12 miner's inches when the orifice may be square. As a result, the head on the center of the opening varies and the value of the miner's inch varies correspondingly from about 35 miner's inches to 43 miner's inches for 1 cubic foot per second. The commonly accepted equivalent is 38.4 miner's inches to 1 cubic foot per second.

In British Columbia the legal value of the miner's inch is the

flow through an orifice 2 inches high × ½ inch wide, made in a 2-inch plank, the head on the top of the opening being 7 inches; this gives a pressure head on the center of the opening of 8 inches (Fig. 163). As defined by law, 35.7 miner's inches are equivalent to 1 cubic foot per second.

Variations in the form of the orifice affect the accuracy of the measurements only to a small extent. To measure small flows the height of the orifice may be only 1 inch; for larger flows the height may be 2, 3, 4, or 5 inches. To obtain accurate measurements the jet coming through the orifice must touch only the upstream edges and clear the downstream edges, so as to discharge freely into the air. Usually it is desirable to make the

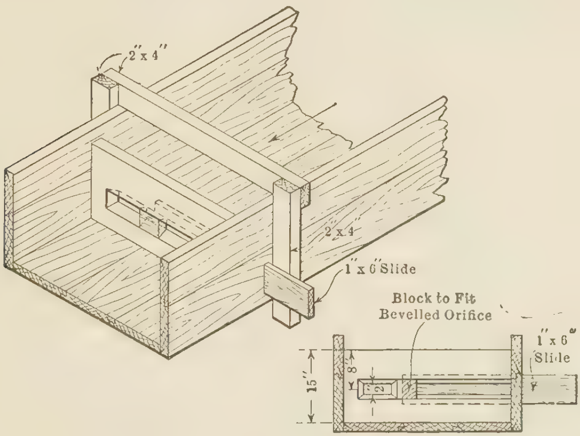


Fig. 163.—Miner's inch board set in a flume box.

orifice in a thin plate or to bevel the edges of the board on the downstream side. An orifice formed with sharp square edges in a board 1 inch thick need not be bevelled. In setting the board across a ditch or in a box, a distance equal to at least 2 or 3 times the height of the opening must be allowed on the upstream side of the board, from the lower edge of the orifice to the bed of the ditch or floor of the box, and from the ends of the orifice to the sides of the ditch or box (Fig. 163).

Special Types of Miner's Inch Boxes.—When placed at the point of delivery from a distributary, the installation of a miner's inch device will combine with it the delivery gate and a very desirable feature may be provided to maintain a practically constant head on the center of the opening by the use of an over-

flow spillway crest. The structure is then what is known as the Foote measuring box (Fig. 164 and Plate XX, Fig. A). It consists essentially of a check gate across the ditch, the delivery gate which controls the flow admitted into a basin, the miner's inch orifice, formed in one side of the basin, adjustable in size by a slide, and an overflow spillway formed by a low side of the basin with its crest at a height above the center of the orifice equal to the desired pressure head. With this device, the flow admitted into the basin is approximately regulated by the delivery gate and the excess flow passes over the spillway crest back into the ditch. The structure is formed of a short flume section divided by a parallel overflow spillway wall into two compartments. The main compartment is a rectangular channel in line with and

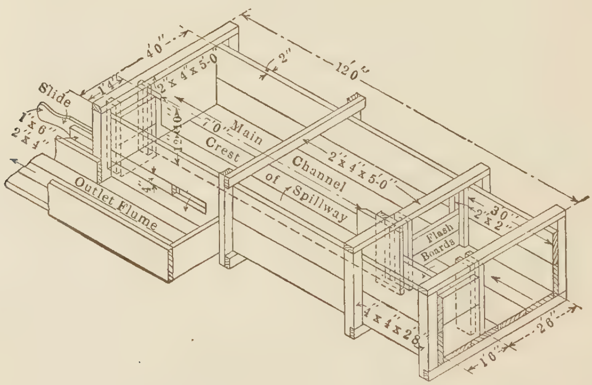


Fig. 164.—Foote's miner's inch measuring and delivery box. (Bull. 247, Agr. Exp. Sta., Univ. of Calif.)

of about the same cross-sectional area as the supply ditch with a check gate near the inlet. The smaller compartment forms the basin from which the water is taken out through the miner's inch orifice. The flow from the supply ditch is admitted into this basin through the delivery gate placed just upstream of the check gate. The installation of this device will require at least about 9 to 12 inches difference in elevation between the full supply water level in the supply ditch and the water level at the orifice outlet. The accuracy of the automatic regulation of head will depend on the length of the spillway crest. Fig. 164 shows the design of a box experimented with at the University Farm at Davis. The tests show that with the water level maintained exactly at the same level as the spillway crest, with discharges

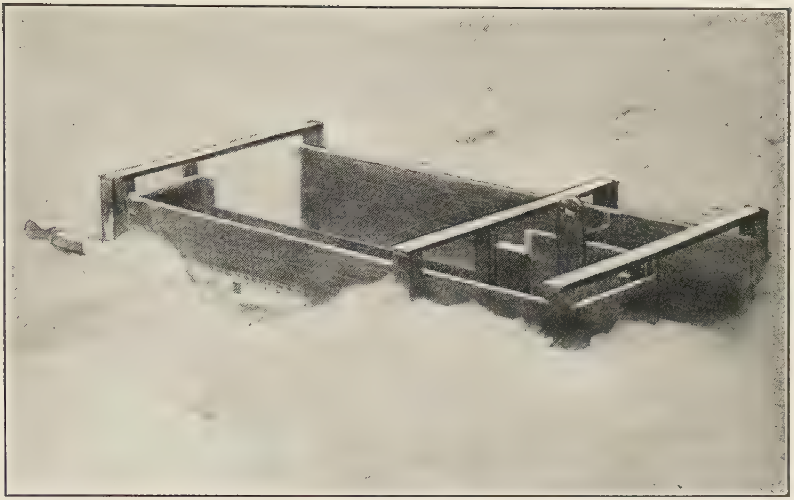


FIG. A.—Small miner's inch box with overflow spillway.

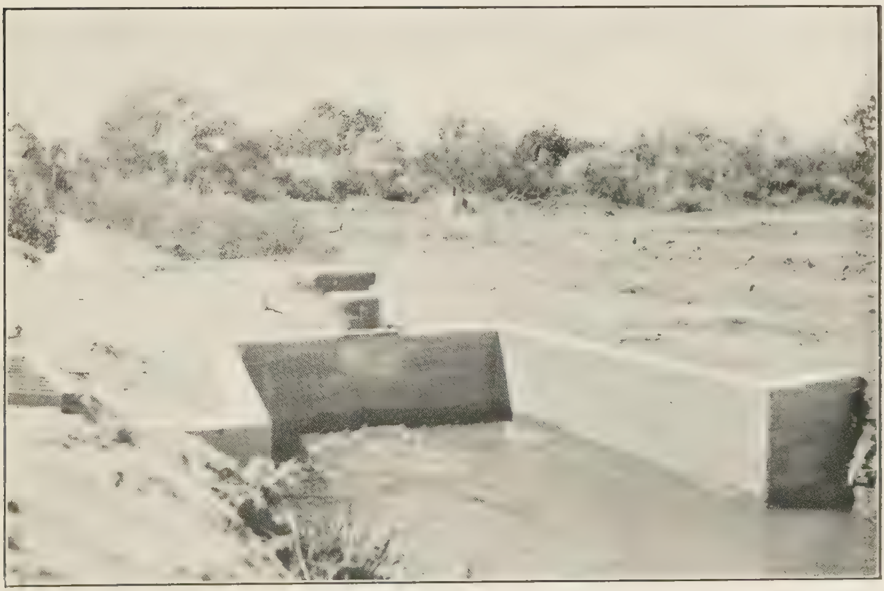


FIG. B.—Thirty second foot Venturi meter installed on lateral Salt River Project, Ariz.

(Facing page 398)

PLATE XX.

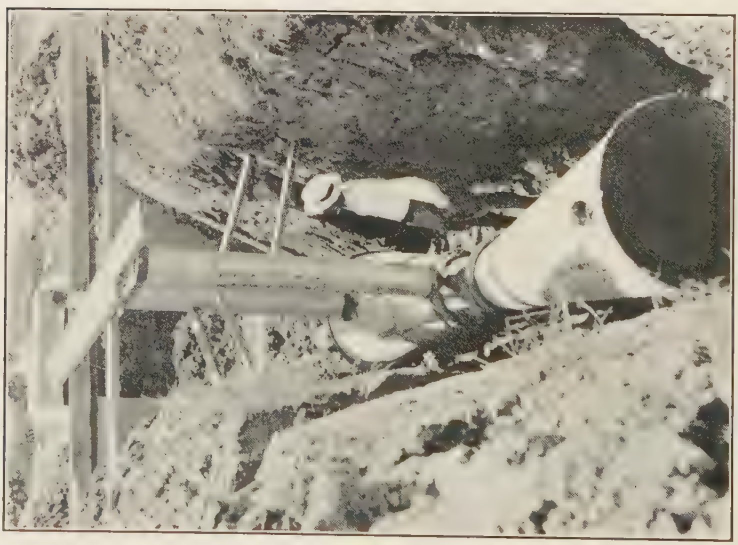


FIG. C.—Installing 15 sec. ft. Venturi meter on lateral of Imperial Water Co., No. 1. Imperial, Calif.

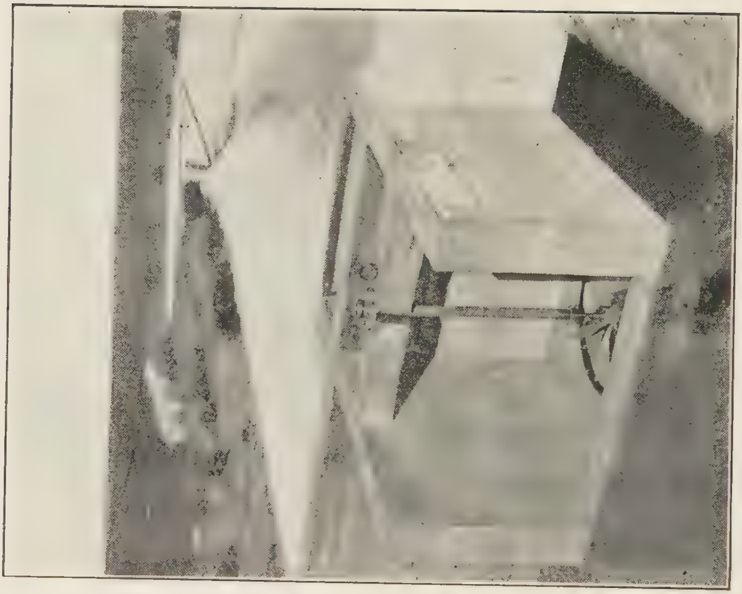


FIG. D.—Downstream view of 39-inch Grant Michell meter Mildura, Australia.

ranging from 20 to 150 miner's inches, the actual flow was about 4 per cent. less than the miner's inch measurements gave.

The Foote box has been used only to a very limited extent, but the same principle of head regulation by a spillway overflow crest has been used on delivery boxes from gravity pipe lines under no pressure. These have been used extensively on several systems in Southern California; also on a system in British Columbia, and have been described in the discussion of gravity pipe-line distribution systems in the preceding chapter. Fig. 154 shows such a box, which was reproduced at the University Farm at Davis, and on which a series of tests for accuracy show that the actual flow was about 1 per cent. greater than the miner's inch measurements gave.

Submerged Orifices and Gate Openings.—The measuring device may be an orifice of a fixed size or an orifice adjustable in size, in which case it is usually a gate opening.

When an orifice of a fixed size is used, the device usually consists of a sharp-edged rectangular orifice formed in a board, which is either placed in a vertical position directly across the ditch or is part of an orifice box. The device will only measure the rate of flow, and is usually placed a short distance below a delivery gate, with which the flow is regulated.

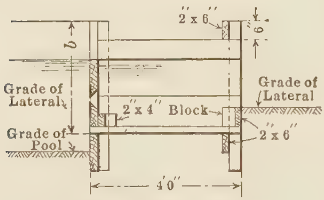
The dimensions of the orifice are controlled by the desired capacity and the amount of fall available. It is usually made from 3 inches to 1 foot in height, and 1 to 4 feet in width. The dimensions of the orifice board must be made sufficiently large to obtain complete contraction and a small velocity of approach. Practically full contraction is obtained when the distance from the edges of the orifice to the sides and bed of the channel is equal to 2 or 3 times the height of the orifice. To obtain submergence the board or box must be installed to place the orifice well below the outlet water level. To obtain a velocity of approach sufficiently small that it may be neglected and not appreciably affect the accuracy of the measurement, the cross-sectional area of the approach channel should be at least about 6 times that of the orifice. The device must be installed sufficiently far downstream from the delivery gate to obtain a pool of fairly still water on the upstream side of the orifice.

To obtain an accurate measurement of the rate of flow, the upstream and downstream water level must be measured where the water is comparatively still. The upstream gauge is usually

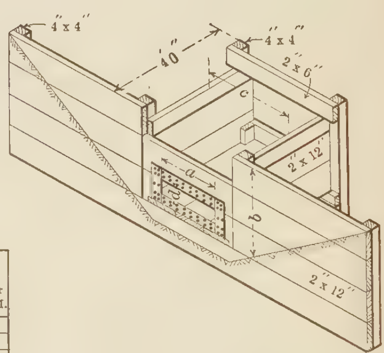
placed on the upstream face of the board on one side of the orifice and the downstream gauge is placed on one of the side walls or wing walls of the box, at least 1 to 2 feet away from the orifice wall.

Fig. 165 shows the standard design for submerged contracted orifices of the U.S. Reclamation Service. For this type of device, with sharp edges and full contraction, the coefficient of discharge in the formula $Q = CA\sqrt{2gH}$ is usually between 0.61 and 0.63 (see Vol. II, Chapter III, page 72).

A submerged orifice of adjustable size is usually the gate open-



Typical Longitudinal Section



Dimensions & Lumber for Submerged Rectangular Orifices

Size of Orifice			Headwall	Floor	Approx.
Height "d" in Ft.	Length "a" in Ft.	Area in Sq. Ft.	Height "b" in Ft.	Width "c" in Ft.	Quantity of Lumber in Ft. B.M.
	1.00	.25	3.0	2.0	150
0.25	2.00	.50	3.0	3.0	170
	3.00	.75	3.0	4.0	185
	1.00	.50	3.0	2.0	150
	1.50	.75	3.0	2.5	160
0.50	2.00	1.00	3.0	3.0	170
	2.50	1.25	3.0	3.5	175
	3.00	1.50	3.5	4.0	210
	1.33	1.00	3.0	2.0	150
	1.67	1.25	3.0	2.5	160
0.75	2.00	1.50	3.0	3.0	170
	2.33	1.75	3.5	3.0	190
	2.67	2.00	3.5	3.5	200

Fig. 165.—Standard submerged orifice box. U. S. Reclamation Service.

ing in a delivery gate structure. The single gate structure is then a vertical gate placed in a rectangular channel formed of a floor and two side walls, with wings, and cut-off walls at the inlet and outlet. The gate as usually placed forms an opening in which the two sides and bottom are flush or nearly flush with the sides and floor of the box. The opening has then no contraction on the sides and bottom. In some cases a low gate sill is placed which produces partial contraction on the bottom. Fig. 166 shows a standard form of delivery gate, 3 feet wide, used by the Yolo Water and Power Co. of California to deliver heads as great

as 20 cubic feet per second. The structure must be either installed sufficiently low or a tail board placed at the outlet of the box to insure complete submergence of the opening. Without the tail board and for the capacity stated above, the floor should be placed about 18 inches or 2 feet below the outlet water level. This method of measurement, while it does not give results as accurate as those obtained with the fully contracted orifice, is quite extensively used because it saves the expense of the additional orifice box and produces a smaller total loss in head. The

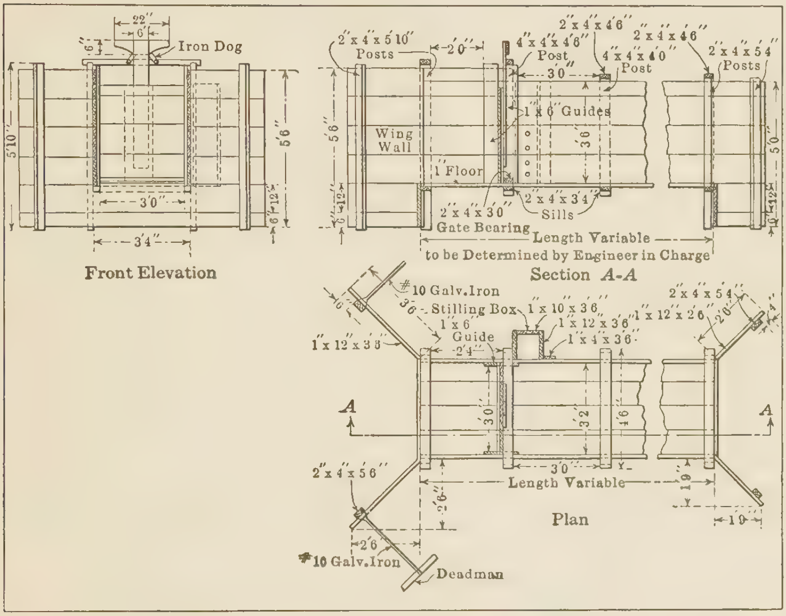


FIG. 166.—Delivery and measuring gate. Yolo Water & Power Co., Calif.

accuracy of the measurements is limited by the fact that the coefficient of discharge is not as well known as for a sharp-edged fully contracted orifice, and also because the coefficient of discharge apparently varies considerably with the ratio of the width to the depth, with the extent of submergence and with the effective head. The variation in coefficient of discharge is greater when no correction is made for the velocity of approach, because for the same sized orifice and same head as measured by the difference in water levels, a wide variation in the velocity of approach may be obtained, depending on the depth of submergence and on

the place of measurement of the inlet and outlet water levels. For these reasons it will usually be desirable to use coefficients obtained by calibrating the type of structure for the conditions of flow under which it is to be used. A rather limited number of experiments have been made to determine the coefficient of discharge. Merriman states that with contraction suppressed on the ends and bottom, the coefficient of discharge is probably about 0.75. Experiments made by the Yolo Water and Power Co., California, give an average coefficient of discharge for their standard headgate of about 0.73. The experiments made by C. B. Stewart at the University of Wisconsin on short tubes, 4 feet square, with submerged discharge, show that for the shortest tubes used, about 4 inches in length, which gives practically orifice condition, and with suppressed contraction on sides and bottom, a coefficient of discharge of about 0.77 was obtained (see Vol. II, Chapter III, page 75). The most extensive experiments and those most directly applicable to the conditions obtained in practice are those made on lateral headgates in Punjab, India. These experiments gave the following coefficient of discharge:

$$C = 0.720 + 0.0074w$$

in which w = width of gate opening in feet (see Vol. II, Chapter III, page 72). This value was obtained from measurements of the inlet and water levels sufficiently far upstream and downstream from the gate to minimize the effect of velocity of approach. It may be taken as the most reliable general value thus far obtained. The formula of discharge is then:

$$Q = (0.720 + 0.0074w)A\sqrt{2gH}$$

This formula was found to be not applicable with the same degree of accuracy, when the effective head was less than 0.25 feet, unless the velocity of approach is negligible.

Submerged Short Tube Delivery Gate.—The delivery gates of the pipe or box culvert type may be used for the approximate measurement of water as well as the regulation of flow when the device is rated so that the coefficient of discharge is known for the conditions of flow. There will seldom be sufficient fall to obtain free discharge at the outlet; it will therefore be usually desirable to place the structure so that the outlet is entirely submerged. The flow will then be that of a submerged tube when the inlet

opening is fully opened, and will be intermediate between the flow of a submerged tube and submerged orifice when the inlet is partly opened. For the submerged tube condition, the coefficient

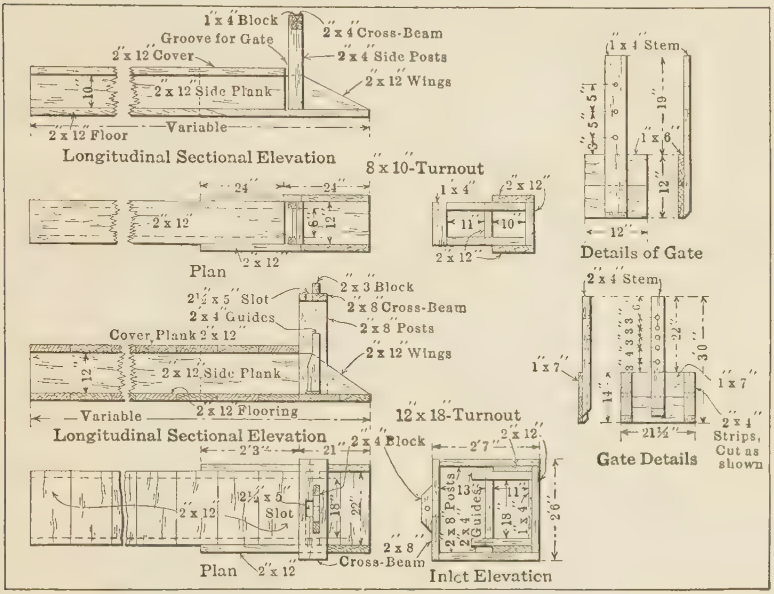


FIG. 167.—Tube delivery gates. Williston Project, N. D.

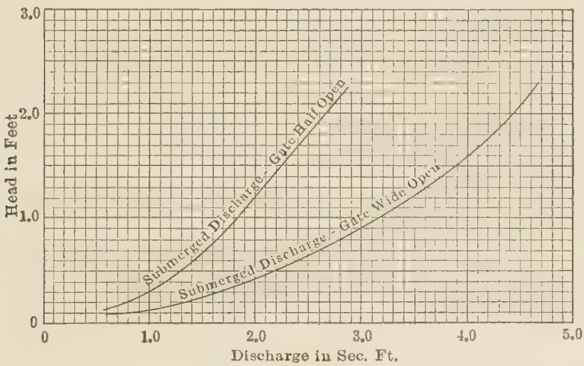


FIG. 168.—Rating curves for 8 in. by 10 in. turnout. Williston Project, N. D.

of discharge in the formula $Q = CA\sqrt{2gH}$ will depend largely on the form of the inlet and the length of the tube. For the condition intermediate between submerged orifice and submerged tube

the coefficient of discharge will depend also on the size of the gate or inlet opening.

The rates of discharge for the common type of box culvert delivery gate or takeout has been determined by experiments made by the Reclamation Service on two sizes of takeouts used on the Williston project, North Dakota. The designs of the takeouts are shown by Fig. 167 and the rating curves by Figs. 168 and 169. A study of the results shows a variation seldom larger than 5 per cent. from the quantities given by the curve. The coefficient of discharge for the inlet fully opened averages very close to 0.70 for the smaller turnout 8 × 10 inches in cross section, and about 0.78 for the larger turnout 12 × 18 inches in cross section. These curves and coefficients should be used for turnouts of the same

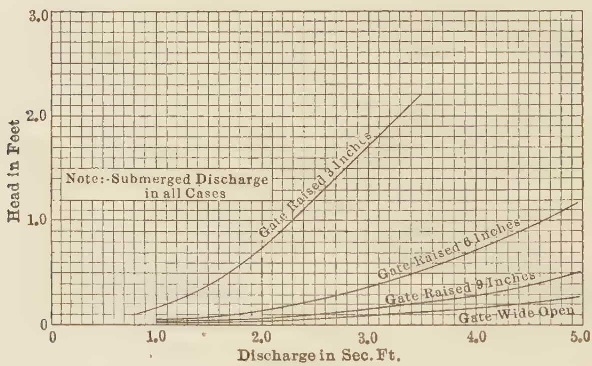


FIG. 169.—Rating curves for 12 in. by 18 in. turnout. Williston Project, N. D.

type and about the same relative proportions of length to size of opening. For approximate results the coefficient of discharge for standard short tubes (Vol. II, Chapter III, page 71) may be used.

SPECIAL DEVICES

Kennedy Gauge Outlet (Fig. 170).—This device was originated and patented by R. G. Kennedy, formerly Engineer in Chief of Irrigation, in Punjab, India. It consists of a conduit formed of a bell-mouth, cast-iron inlet, contracting to a throat and connected to a conical shaped outlet, made usually of sheet metal. The throat is constructed to form a groove (M_2–M_3) around the jet of water, and is connected to an upright vent shaft K_1, extending above the water level with its upper end open to the air. The

jet of water at the throat is therefore surrounded with air, which causes the flow to depend not on the difference in elevation between the water surfaces at the inlet and outlet, but on the square root of the head on the center of the throat opening (H_0). The flows corresponding to different elevations of water level are graduated on the face of the vent shaft. To operate the device, a certain minimum difference in water levels, H_1, equal to about $\frac{1}{4}$ or $\frac{1}{5}$ of H_0, must exist or the water will back up at the throat in the air-shaft K_1. Provided this minimum difference in elevation between the inlet and outlet water level is obtained, the discharge is independent of the outlet water level. The chief features of the device are that the flow is dependent on a comparatively large effective head and is independent of the outlet water level. Because of the large effective head, a variation in the inlet water level will produce a change in the flow much smaller than if the discharge depended on the difference in elevation

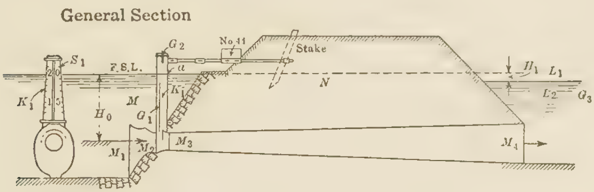


FIG. 170.—Kennedy gauge outlet.

between inlet and outlet water levels as in a submerged orifice. For instance, if delivery is to be made through a submerged orifice of a given size, where the difference in elevation between outlet and inlet water levels may increase from 6 to 12 inches, the flow will increase in the proportion of $\sqrt{6}$ to $\sqrt{12}$, or about 40 per cent.; while if a Kennedy gauge outlet is used in which the effective head is 24 inches, a rise in the inlet water level of 6 inches will increase the flow in the proportion of $\sqrt{24}$ to $\sqrt{30}$ or about 12 per cent. The device is provided with a slide gate G_1 to shut off the flow when needed; thus serving the double purpose of a measuring device and of a delivery structure.

The Kennedy gauge outlet produces conditions which make it similar to a free discharge orifice, of a fixed size, under a comparatively large pressure head. While the throat or orifice has a fixed cross section, the slide gate could be used to vary the size of the opening and thus deliver partial flows.

The main advantages of the device are:

First.—It can be used where only a small head is available for the installation of a device.

Second.—The flow is not affected by variations in the outlet water level.

Third.—A variation in the inlet water level produces a relatively small variation in flow.

The main disadvantages are:

First.—It is not specially designed to be used for fractions of its full capacity.

Second.—The cost will probably be excessive for general use. The conditions most favorable to its use would be obtained where it is desired to deliver a relatively constant flow from a canal in which relatively large variations in water level occur. The device measures the rate of flow only, and will therefore require an automatic register where a record of the volume delivered is desired, in which case one of the types of meters described further will usually be preferable.

The device was manufactured in the United States for a short time by W. N. Matthews & Bros., of St. Louis, Missouri, in sizes ranging from the smaller size of 0.55 to 0.65 second-feet capacity to the larger size from 4.40 to 5.20 second-feet capacity. The prices ranged from $32 for the smaller size to $54 for the larger size. As far as known to the writer, their use in the United States has been limited to a few installations for experiments and demonstrations.

Venturi Meter.—The Venturi meter is a short tube formed of a truncated reducing conical inlet, connected by a short throat section to a truncated expanding cone outlet. The measurement of the rate of flow through the tube is based on the fact that for every rate of flow there is a corresponding difference between the pressure head at the inlet and that at the contracted throat. This is expressed by the following equation of flow, derived by application of Bernouilli's theorem (Fig. 171):

$$Q = \frac{CA_1A_2}{\sqrt{A_1^2 - A_2^2}} \sqrt{2g(h_1 - h_2)}$$

Where A_1 = inlet area.

A_2 = throat area.

h_1 = pressure head at inlet.

h_2 = pressure head at throat.

C = coefficient of discharge = 0.96 to 0.99.

The Venturi meter, invented by Clemens Herschel in 1887, is the result of the practical application for the measurement of water of the properties of converging tubes, based on experimental work made by G. B. Venturi in 1797.

The meter as commercially manufactured is equipped with a

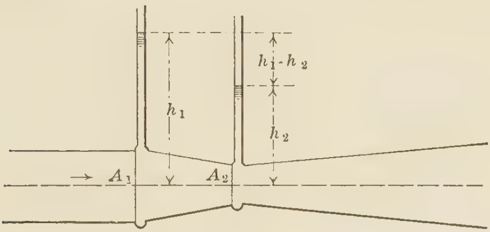


FIG. 171.

recording mechanism, which not only measures the rate of flow but also registers the quantity of water discharged. It has been extensively used for the measurement of flow in pipes, and is now made in a special form, designed for use on irrigation systems. When used on a pipe line, the tube is inserted to form part of the

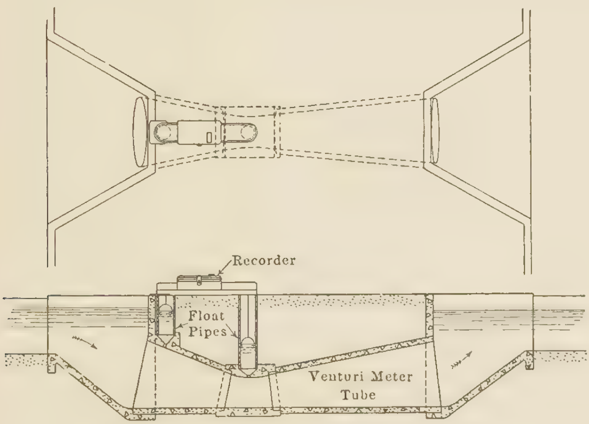


FIG. 172.—Venturi meter for irrigation canals.

pipe line and the recording mechanism consists essentially of a mercurial U-tube, one branch connected with the inlet and the other to the throat and of the clockwork and gear which revolve a recording drum. The variations in rate of flow give a resultant motion to floats placed in the mercurial tube, which transmitted

through gearing and combined with the clockwork motion produce an integrating record on the cylindrical drum.

The specia' form developed for use on irrigation canals is shown in Fig. 172 and Plate XX, Figs. B and C. The tube is made of a cast-iron throat with conical pieces made of galvanized iron, or cast in concrete, or made of wood staves. The tube is placed directly in the ditch. At the inlet and the throat are connected vertical pipes or float chambers, in each of which is placed a float. The resultant motion of the float, corresponding to variations in the rate of flow, are transmitted to the recording device and combined with the clockwork motion gives a record on a chart of the rates of flow and the quantity of water passed through the meter in a given time.

The present approximate prices of Venturi irrigation meters delivered to principal points in the West are as follows:

Capacity in second-feet......	5	10	15	20	30	40	50	75	100	125
Cost in dollars.............	113	132	150	168	200	231	263	374	405	477

The well-shaped tapered entrance and outlet results in a very small friction loss. The manufacturers guarantee that the loss of head will not exceed 4 inches when operated at their maximum rated capacities. The accuracy of Venturi meters compares favorably with that of any other device. A test for accuracy made on a standard 15-second-foot capacity Venturi irrigation meter by a board of engineers appointed by the project manager of the Salt River project, Arizona, with flows ranging from 2.96 cubic feet per second to 16.86 cubic feet per second, showed that the meter readings average 1.67 per cent. less than the weir readings and 2.72 per cent. larger than current meter measurements.

The main advantages of the Venturi irrigation meter are:

First.—It measures the rate of flow and registers the quantity of water passed through in a given time.

Second.—Its installation produces only a small loss of head.

Third.—It is equally well adapted for small and large flows.

Fourth.—It can be used with silty or muddy water.

Fifth.—The measurement cannot be tampered with or altered.

Sixth.—The accuracy is equal or greater than that of any other device. The most serious disadvantage is the cost, which may be excessive for extensive use of the meter.

Dethridge Meter (Figs. 173 and 174).—This meter was invented by J. S. Dethridge, Chief Engineer of the State Rivers and Water

Supply Commission of Victoria, Australia. The meter is essentially an undershot wheel placed horizontally in a wooden or concrete structure, which also serves as a headgate. The wheel is formed of a galvanized iron drum on a wooden frame, to which are riveted eight projecting metal vanes or paddles. The structure forms a channel, whose width gives a ⅜-inch clearance between the face of the side wall and the ends of the wheel; and the floor is curved on a radius ¼ inch greater than that of the wheel for a length equal to about ¼ of the circumference. The wheel will then fit in this channel with a clearance of ⅜ inch on the sides

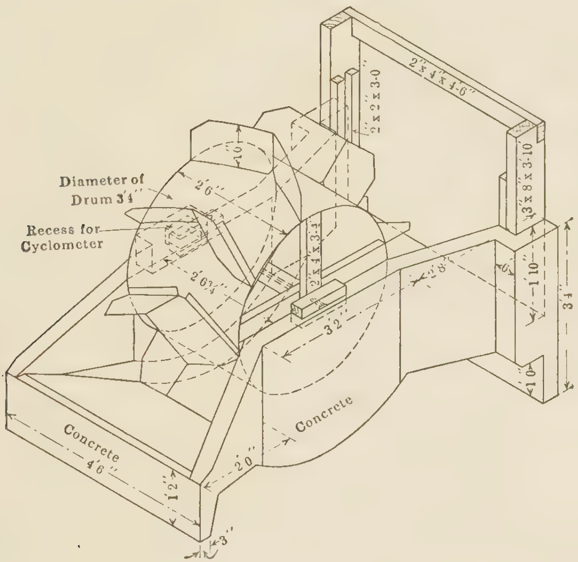


FIG. 173.—Isometric view of Dethridge meter. (Bull. 247, Agr. Exp. Sta., Univ. of Calif.)

and ¼-inch on the bottom. The inlet to the structure is regulated with a gate, and a hinged frame is provided to lock the gate and wheel at the same time when the gate is shut.

The standard size of wheel is 2 feet 6 inches in width, and is designed for a normal capacity of 4 second-feet, which may be crowded to 5 second-feet. The loss in head or fall necessary to operate the meter is small; it ranges from about ½ inch for a flow of ½ cubic foot per second to about 4 inches for a flow of 4 cubic feet per second. The flow of the water causes the wheel to revolve and the revolutions are registered on a Veeder counter. The

water enclosed in a volume equal to that between the outside face of the drum and the outer edges of the vanes is 27.3 cubic feet. This would be the volume passed through the structure for one revolution of the wheel if there were no leakage. The leak-

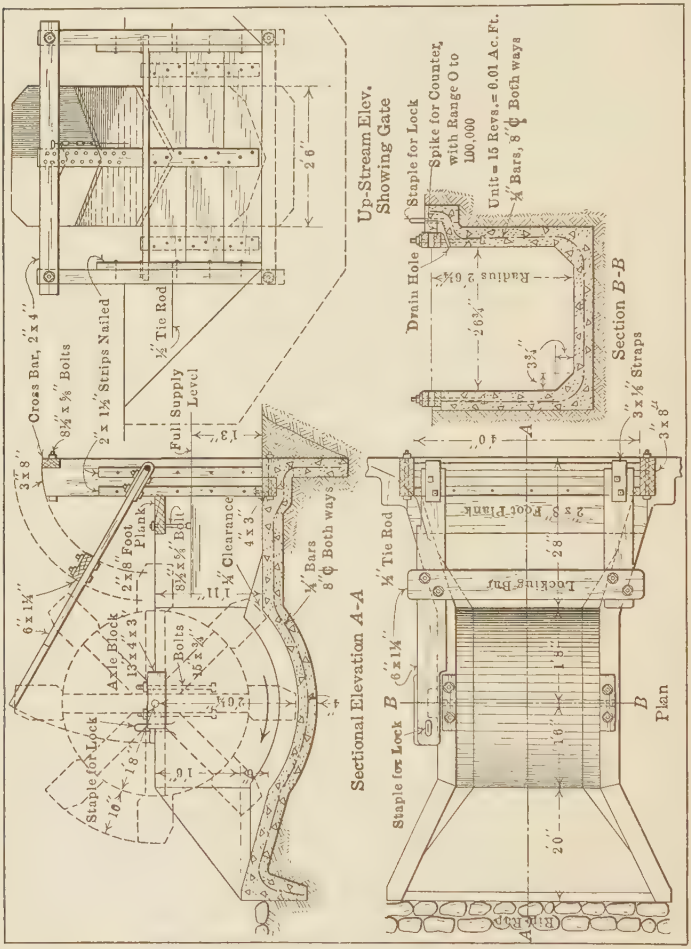


FIG. 174.—Dethridge meter.

age will depend on the difference in upstream and downstream water levels and on the depth of water on the upstream side. The results of tests made at the University Farm at Davis, California, show that the actual volume of water passed through for one revolution was 28.75 cubic feet for a relatively small depth of

water in the channel, 30.4 cubic feet for a greater depth, and 33.0 cubic feet for the maximum depth used.

Plans furnished by the State Rivers and Water Supply Commission of Australia indicate that the structure is placed relatively high in the ditch, to insure a small depth of water in the channel. In these plans the horizontal shaft of the meter is 8 inches above the full supply water level. The tests made at Davis indicate that these conditions are necessary for accurate measurements and that when they are obtained the meter measurements will be sufficiently accurate.

Grant Michell Meter (Fig. 175 and Plate XX, Fig. D).—This meter was originated in Australia, where it has been used to a considerable extent. It consists of a four-bladed fan, fastened to the lower end of a vertical spindle which transmits the revolu-

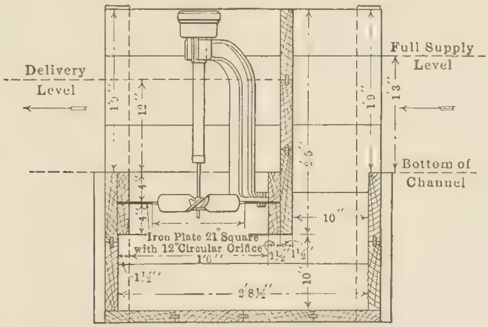


FIG. 175.—Grant Michell 12-in. meter.

tion of the fan to a gear box at the upper end. This mechanism is suspended from a cast-iron bracket over a wrought-iron orifice placed below the canal bed and built or bolted down into the downstream part of a box divided into two parts by a baffle wall open at the bottom and extending above the water surface. The water passes down through the opening and imparts a rotary motion to the fan. The gear box forms the recording device, which consists of a series of dials giving a continuous record in acre inches and fractions or in cubic feet. The fan spindle and gear box are removable and portable and can be used for several boxes. The discharge depends on the size of the orifice plate and on the difference in elevation in the water surface upstream and downstream.

The standard sizes of meters, as denoted by the diameter of the

circular openings, are 12-inch, 18-inch, 21-inch, 39-inch and 66-inch. The corresponding capacities, as given by the manufacturers with 3 inches loss of head, are 1.66, 3.75, 5.83, 16.66 and 50 cubic feet per second. These meters are now made in the United States by Ainsworth & Sons of Denver, Colorado, who quote as follows for the smaller sizes:

12-inch meter, bronze	$35.00;	iron	$30.00
18-inch meter, bronze	45.00;	iron	40.00

These prices include the bracket and orifice plate. The meters will give sufficiently accurate records for considerable variations in the rate of flow or loss in head. This is illustrated by the following tests made on the meters supplied to the State Rivers Commission of Victoria, Australia:

MEAN OF TWO 21-INCH METERS, 1 YEAR IN USE

Number of test	By weir, cubic feet per second	By meter, cubic feet per second	Difference, per cent.
1	4.71	4.45	−5.5
2	3.71	3.60	−3.0
3	2.72	2.52	−7.3
4	1.71	1.65	−3.5
5	1.13	1.07	−5.3
6	0.41	0.43	+4.9

MEAN OF TWO 21-INCH METERS, 2 YEARS IN USE

Number of test	By weir, cubic feet per second	By meter, cubic feet per second	Difference, per cent.
1	4.67	4.35	−6.8
2	3.73	3.55	−3.8
3	2.74	2.54	−7.3
4	1.73	1.64	−5.2
5	1.10	1.05	−4.5
6	0.42	0.43	+2.4

For all rates of flow, except the lowest in each series, the registered flow is smaller than the flow measured by the weir, averaging about 5 per cent. less for the first series, and 5.5 per cent. for the second series. The manufacturers state that the meters are designed to register about 3 per cent. low to avoid overcharging consumers.

For flows which are only a small fraction of the capacity of the meter, the registered flow will be smaller than the actual flow; this is as might be expected and is due to the retarding effect of friction on the movement of the wheel.

The most serious objections against its general use are the comparatively high cost, the possibility of obstruction by sticks, weeds or other similar material transported by the water and probably the deposition of silt with waters carrying much silt. A screen may be used at the inlet to the structure, but may require frequent removal of the weeds collecting on the screen, in order to prevent obstruction of the flow which might result in overflow of the banks of the canal, with the resulting damage.

The Hill Meter (Fig. 176).—This device is very similar in construction and in operation to the Grant Michell meter. It

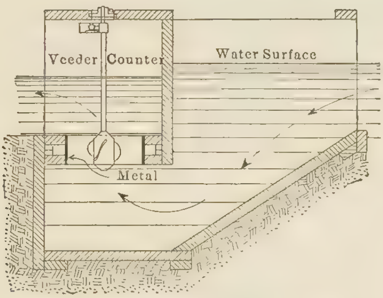


FIG. 176.—Section of Hill meter.

is formed of a short vertical hollow cylinder, through which the water flows and in which is placed a revolving cylindrical head with projecting vanes, fixed to the lower end of a shaft, whose upper end is connected to a counter. The water flows upward through the cylindrical opening, and causes the meter to revolve. The volume of water passed is registered by the counter and can be read in units of 0.001 acre-feet. No special efforts have been made to push the sale or use of these meters; it is estimated that the meter for the 12-inch opening should cost about $10; for larger sizes the cost should not be much greater. The results of thirteen tests made at the University Farm, Davis, California, on a 12-inch meter, indicate that for discharges varying from about 1 to 3.5 cubic feet per second, the registered discharge was 1.3 per cent. less than the actual discharge. Twenty-three tests made by the U. S. Reclamation Service at Boise, Idaho,

with discharges varying from 1.11 to 3.96 cubic feet per second, give an average registered discharge of 2.5 per cent. less than the actual discharge.

This meter is less liable to be obstructed by sticks, weeds or

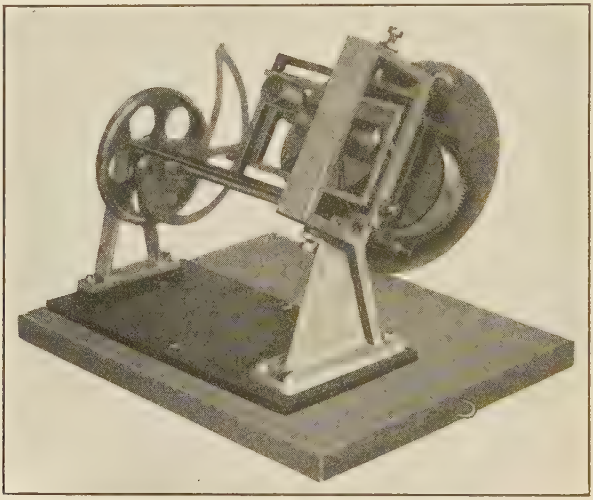


Fig. 177.—Hanna self registering meter. (Hess Flume Co., Denver, Colo.)

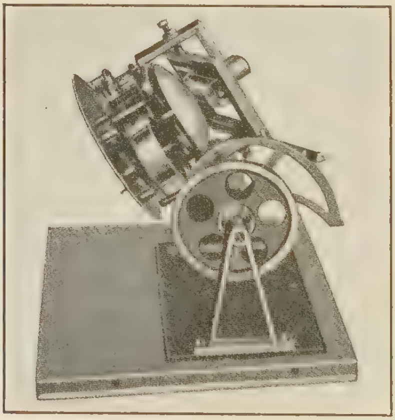


Fig. 178.—Hanna self registering meter. (Hess Flume Co., Denver, Colo.)

similar material transported by the water than the Grant Michell, but is equally liable to cause the deposit of coarser silt or sand in the part of the box below the opening.

Hanna Self-registering Meter (Figs. 177 and 178).—This meter, invented by F. W. Hanna, is manufactured by the Hess

Flume Co., of Denver, Colorado. The retail price is $50. The meter is a special type of automatic register, which, when used in connection with a rating station or flume, a weir (Fig. 179), a submerged orifice, or a free discharge orifice, will register the quantity of water passed on a counter, directly in acre-feet. The meter consists of a clock, a spherical planimeter attachment and

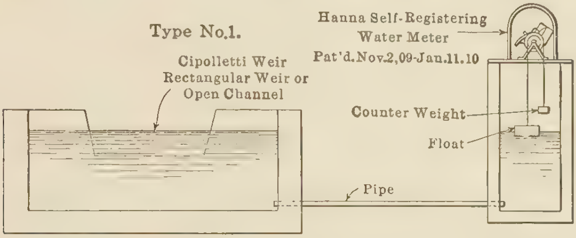


FIG. 179.—Hanna meter used in connection with Cippoletti weir. (Hess Flume Co., Denver, Colo.)

a float driven cam. The clock drives a section of a sphere, which in turn drives a cylindrical drum, mounted on a shaft, which carries a worm gear to drive a recording device. The cam is connected to the same axis as the float wheel, and bears against an arm extending from the frame, which carries the cylindrical drum. The cam is operated by the float and float wheel, and its motion shifts the point of contact between the cylinder and

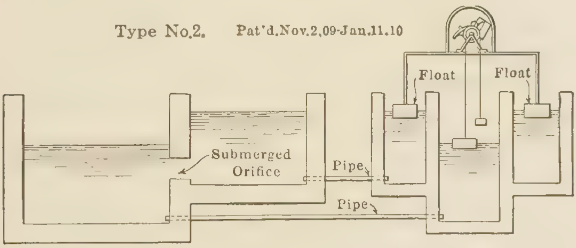


FIG. 180.—Hanna meter used in connection with submerged orifice. (Hess Flume Co., Denver, Colo.)

the sphere, such that the integrated record will correspond to the discharge. The shape of the cam is made to correspond with the discharge curve of the rating station, flume, weir or orifice. For the submerged orifice the meter itself is supported on floats placed in float wells connected with the upstream side of the orifice and the cam is operated by a float placed in a float well connected with the downstream side of the orifice (Fig. 180). The

motion of the cam corresponds then to the difference in water levels.

GAUGES AND AUTOMATIC REGISTERS

Gauges may be divided into two classes:

First.—Those which are graduated in feet and inches or in feet and decimals.

Second.—Those which are graduated to read the flow directly in cubic feet per second.

Gauges of the first class are generally used at rating stations and

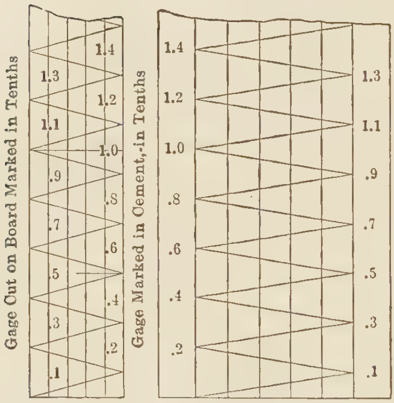


Fig. 181.—Steward water gauges.

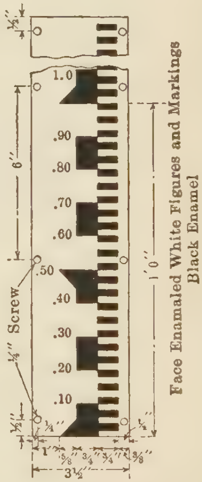


Fig. 182.—Enamelled gauge. Tieton Project, Wash.

flumes and with orifices to read the depth of water. Various forms have been devised to permit accurate reading of the water level. Fig. 181 shows the form of gauges devised by W. G. Steward, Assistant Engineer U. S. Reclamation Service, Boise project, Idaho. One form is for a wooden gauge; the other for a gauge marked or stamped in soft cement. The design formed of sloping and vertical lines gives intersections which permit direct reading to $1/50$ of a foot. For the wooden gauge the design is formed with saw cuts about $1/8$ inch wide and $1/8$ inch deep. An enamelled gauge designed for use on the Tieton project, Washington, is shown in Fig. 182. This gauge is graduated to read directly in tenths and hundredths of a foot.

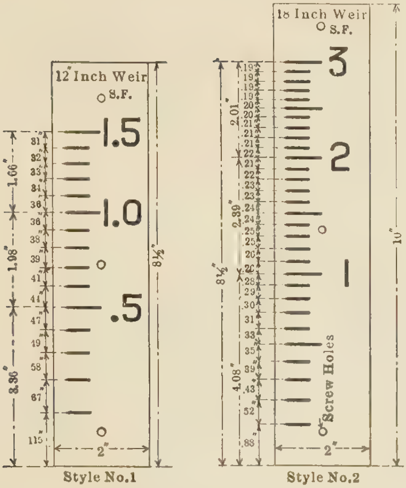


FIG. 183.—Weir gauges graduated in cubic feet per second. Tieton Project, Wash.

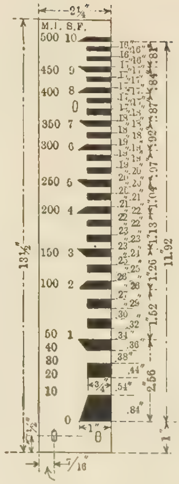


FIG. 184.—Weir gauge for 3 ft. Cippoletti weir. Oakley Project, Idaho.

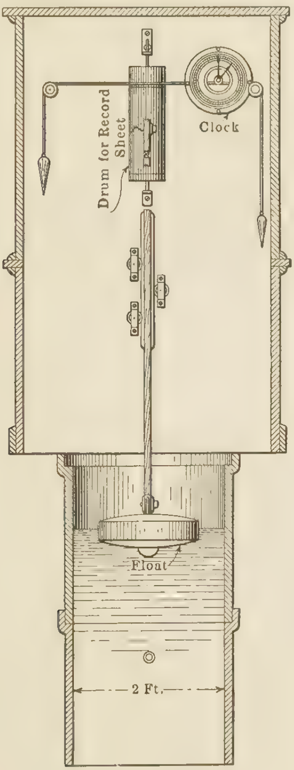


FIG. 185.—Automatic register used on Gage canal. Southern California.

Gauges of the second class, graduated to read the flow directly, are often used for weirs. Fig. 183 shows two styles of enamelled gauges, with black markings on a white background, designed for the Tieton project, Washington. Fig. 184 shows an enam-

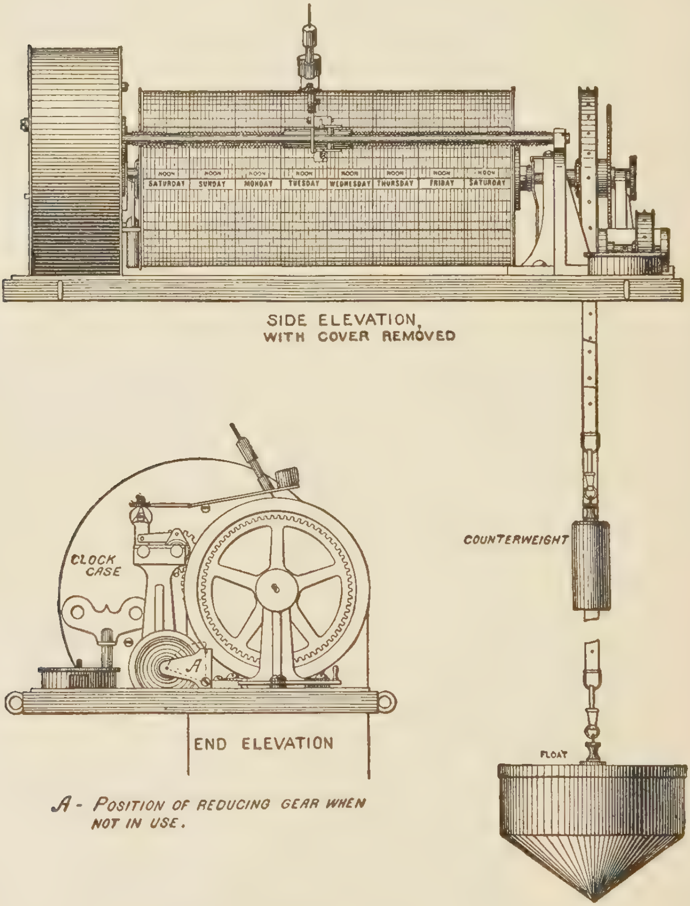


FIG. 186.—Automatic register. (Bull. 86, Office of Exp. Sta., U. S. Dept. Agr.)

elled gage for a 3-foot weir, in which the graduations give the flow in cubic feet per second with the corresponding flow in miner's inches. This style of gauge was made for the Oakley project, Idaho, and cost about 25 cents apiece.

Automatic registers are used at rating stations and weirs when

it is desired to have a continuous record of the variations in water level, with which the corresponding rates of flow and total volume of water in a given time are obtained. There are various makes and types of automatic registers. In general they consist of a clock, a float with counterweight, and a cylinder or drum, to which is fastened a sheet of paper on which the depth of water at different times is recorded by a pencil or pen.

The registers most commonly used in irrigation work can be grouped into two classes. In one class the cylinder is placed vertically and is rotated by a clock which gives it one revolution a week (Fig. 185). The pencil is connected to the float, which is placed in a well or box built on the side of the weir box or rating flume or canal and connected with the water through an orifice. The fluctuations in water level cause rises and falls of the float and corresponding movements of the pencil, which are recorded on the sheet placed on the drum.

The other class of register differs from the first class in that the cylinder is placed horizontally and is rotated by the float instead of by the clock and the pencil is carried parallel with the cylinder by connections with the clock (Fig. 186). In each case the record obtained is the result of two motions which give an irregular line, showing the fluctuations and giving the depth of water at any time.

The cost of registers ranges from about $40 upward, and for that reason they are seldom used for the measurement of water delivered to irrigators. They are, however, of much value in the operation of a system when installed at the head of laterals.

REFERENCES FOR CHAPTER XIII

Carpenter, L. G.—The Measurement and Division of Water—Bull. 27, State Agr. College, Fort Collins, Colorado—1894.

Benton, R. J.—Experiments on Gauging the Supply Entering Rajbahas by Means of the Headgates—Punjab Irrigation Branch Papers No. 8—1897.

Johnston, C. T.—Instructions for Installing Weirs, Measuring Flumes and Water Registers—p. 131 Engineering News—Aug. 29, 1901.

Fleming, B. P.—The Measurement of Water for Irrigation—Bull 53, Wyoming Experiment Station, Laramie, Wyo.—1902.

Fortier, S.—Farmers' Weirs—Bull. 34, Montana Agricultural Experiment Station, Bozeman, Mont.—1902.

A Diagram for Computing the Flow of Water Over Cippoletti Weirs—p. 157 Engineering News—Feb. 8, 1906.

KENNEDY, R. G.—On the Distribution of Water for Irrigation by Measurement—Punjab Irrigation Branch Paper No. 12—June, 1906.

HORTON, R. E.—Weir Experiments, Coefficients and Formulas—Water Supply Paper No. 200—U. S. Geological Survey, Washington, D. C. —1907.

MEAD, E.—An Australian Irrigation Ditch Water Meter—p. 346 Engineering News—March 26, 1908.

A Huge Venturi Meter Registering 625 Cubic Feet Per Second, Divi Island Pumping Plant, Madras, India—p. 577 Engineering News—Nov. 25, 1909.

SANFORD, G. O.—The Measurement of Water to Farm Units—p. 91 Operation and Maintenance Conference, Powell, Wyoming—U. S. Reclamation Service—Nov., 1909.

STEVENS, J. C.—Experiments on Small Weirs and Measuring Modules—p. 171 Engineering News—Aug. 18, 1910.

STEVENS, J. S.—Hydrometry as an Aid to Successful Operation—p. 314 Amer. Soc. Civ. Engr., Vol. LXXI—1911.

GIBB, A. S.—Gibb's Module—Punjab Irrig. Branch Paper No. 13—Jan., 1911.

HANNA, F. W.—Irrigation Management—p. 290 Engineering News—Feb. 15, 1912.

GIBB, A. S.—Use of the Cippoletti Weir for Ascertaining the Discharges of Irrigation Water Courses—Punjab Irrigation Branch Paper No. 14—March, 1912.

STEWARD, W. G.—Water Measuring Devices—Journal of Idaho Society of Engineers—June, 1912.

STEVENS, J. C.—A New Continuous Water Stage Recorder—p. 166 Engineering News—July 25, 1912.

STEWARD, W. G.—Some Methods of Measuring Irrigation Water on U. S. Reclamation Service—p. 215 Engineering and Contracting—Aug. 21, 1912.

LYMAN, R. R.—Measurement of Flowing Streams—Bull. 5, Utah Engineering Experiment Station, University of Utah, Salt Lake City, Utah—Sept., 1912.

WINSOR, L. M.—Measurement and Distribution of Irrigation Water—Cir. 6, Utah Agriculture College, Logan, Utah—1912.

STEWARD, W. G.—Measuring and Recording Devices for Irrigation Systems—p. 390 Engineering News—Aug. 29, 1912.

HOSEA, R. G.—Automatic Register, Water Measuring Device—Engineering Record—Sept. 21, 1912.

LYMAN, R. R.—Why Irrigation Water Should be Measured—p. 722 Engineering News—Oct. 17, 1912.

Gibb's Module for Irrigation Works—Engineering—Jan. 17, 1913.

A New Module or Constant Discharge Device for Irrigation Works—p. 502 Engineering News—March 13, 1913.

WRIGHT, A. E.—Methods and Devices for Measuring Water for Irrigation—Engineering and Contracting—May 21, 1913.

STEWARD, W. G. AND LONGWELL, J. S.—Experiments on Weir Discharge—p. 1045, Vol. LXXVI, Trans. Amer. Soc. Civ. Engr.—Dec., 1913.

Kneale, R. D.—Measurement of Water—Cir. 24, Montana Agriculture College, Bozeman, Mont.—1913.

Measurement of Irrigation Water with Tables (59 pages)—U. S. Reclamation Service, Washington, D. C.—1913.

Waller, O. L.—How to Measure Water—Cir. 1, Engineering Division of the Extension Dept., State College of Washington, Pullman, Wash. —Feb., 1914.

Lyman, R. R.—Measurement of the Flow of Streams by Approved Forms of Weirs with New Formulæ and Diagrams—p. 1189, Vol. LXXVII, Trans. Amer. Soc. Civ. Engr.—Dec., 1914.

Some Measuring Devices Used in the Delivery of Irrigation Water—Bull. 247, Agriculture Experiment Station, University of California, Berkeley, Cal.—1915.

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